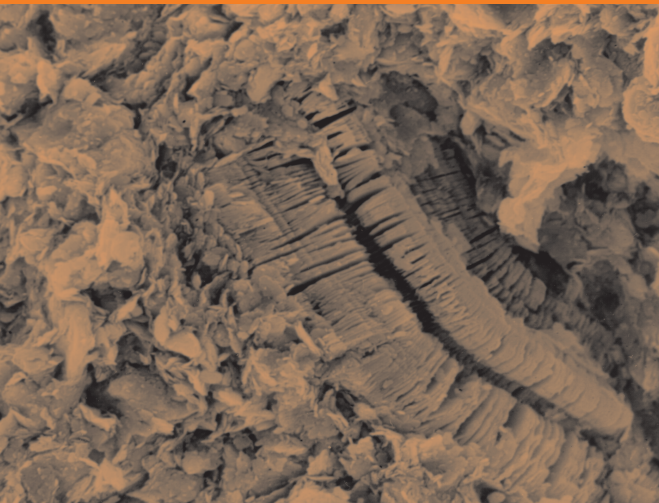


CHARACTERISATION AND ENGINEERING PROPERTIES OF NATURAL SOILS

volume 3



**T.S. TAN
K.K. PHOON
D.W. HIGHT
S. LEROUAIL**
editors

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BALKEMA – Proceedings and Monographs
in Engineering, Water and Earth Sciences

Characterisation and Engineering Properties of Natural Soils

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Volume 3



Taylor & Francis

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LONDON / LEIDEN / NEW YORK / PHILADELPHIA / SINGAPORE

CRC Press
Taylor & Francis Group
6000 Broken Sound Parkway NW, Suite 300
Boca Raton, FL 33487-2742

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Version Date: 20150311

International Standard Book Number-13: 978-0-415-88951-3 (eBook - PDF)

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Preface

The first international workshop on Characterisation and Engineering Properties of Natural Soils was held in Singapore between 2 and 4 Dec 2002. The major goal of this workshop was to compile the existing body of knowledge on natural soils into a single authoritative reference text. The resulting text (two volumes) contains three overview papers pertaining to engineering geology, mechanical behaviour, and engineering implications and thirty-four invited papers covering marine clays, estuarine clays, lacustrine clays, stiff clays, sands and other cohesionless soils, residual and other tropical soils, and weak rocks. Both volumes were well received by the wider geotechnical engineering community and they have made an impact beyond the expectations of the editors.

There were two reasons for continuing this enterprise. First, we were alerted to the existence of distinguished works carried out on other soil types after the completion of the first workshop. Given our original goal of covering all natural soil types that were well documented, there was a compelling reason to fill this gap. Another reason was to attempt to bridge two bodies of work that are intimately related but developed somewhat separately, namely soil characterisation and geostatistics. The characterisation of natural soils primarily focuses on sampling, testing, and the development of physical models that can account for complex effective stress-strain-time responses to external actions. Topics such as microstructure, anisotropy, viscosity, partial saturation, and weathering are under active research. However, natural variability is less well studied because it cannot be adequately reproduced at micro- and meso-scale experiments. However, the effect of natural variability at engineering scale is well known. It is important to make clear that natural variability occurs at all scales, but the term “natural variability” usually applies to only scales measurable by in-situ tests (exceptions could be shear wave velocity and comparable small-strain non-intrusive measurements). There is currently a knowledge gap between our physical understanding of microstructure and how statistical variations at micro-scale impact engineering-scale systems. It is anticipated that a cross-pollination of ideas between these two domains would be productive and would advance our state-of-knowledge considerably. It is with these reasons in mind that in this second effort, a deliberate attempt was made to include a session on natural soil variability.

The present proceedings are numbered Vols. 3 and 4 because they form an integral part of the entire series. It contains three overview papers on laboratory testing, in-situ testing, and characterisation of natural soil variability. Less well documented soil types such as organic clays and peats, loess, volcanic soils, residual soils, methane hydrate soils, and recent sediments are covered by fifteen papers. The remaining papers cover: alluvial and marine clays (6), highly fissured clays (1), boulder clay (1), silts (1), and natural soil variability (4). Following the tradition of the first workshop, authors were invited under consultation with the advisory committee chaired by Dr David Hight. The length and quality of the papers speak volumes on the tremendous dedication and efforts invested by all the invited authors. It is worth reiterating that the major part of the credit for this entire series belongs to the authors.

Volumes 3 and 4 represent the culmination of a long journey we began (perhaps foolishly) about six years ago. The original idea grew out of a series of conversations we had with Dr David Hight when he was visiting National University of Singapore as our Distinguished Visiting Professor. Dr Hight and Professor Leroueil are undoubtedly the main driving forces behind the entire enterprise and their relentless pursuit of perfection and quality is instrumental to the successful completion of this humungous and unique series (68 papers totaling 2840 pages in Vols. 1 to 4 involving 152 authors from 24 countries). In particular, the irrepressible passion and dedication of Professor Leroueil have sustained us in the second leg of this journey. We are also grateful for the constant support and timely assistance rendered by the advisory and organizing committees, the technical reviewers (Siang-Huat Goh, Eng-Choon Leong, Chun-Fai Leung, Nicholas Shirlaw, David Toll) and editorial reviewers (Krishna Bahadur

Chaudhary, Cheng-Ti Gan, Chee-Wee Ong, Czha-Yheaw Tan, Kar-Lu Teh, Sindhu Tjahyono, Haibo Yang), and the secretariat (Ms Nur Azlinawaty A. R. and Hui-Leng Lim).

Finally, we gratefully extend our acknowledgment for the friendship, patience, understanding, and encouragement from all participants of this enterprise that made the entire endeavour enjoyable and satisfying.

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K.K. Phoon (*Secretary*)

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Overview papers

Laboratory testing of natural soils – Some factors affecting performance

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ABSTRACT: This paper presents a review of current geotechnical laboratory testing practice for natural soils. It considers the apparatus, instrumentation and procedures used for testing, and draws attention to those factors that have been observed by the authors to lead to variations in soil testing.

1 INTRODUCTION

Geotechnical laboratory testing of natural soils (as distinct from chemical testing for contaminated land) can trace its origins back at least to the 19th century when Collin (1846) used the shear box to measure undrained shear strength. But it was the early 20th century before there was a significant growth of techniques with, for example, Terzaghi developing the oedometer apparatus, Casagrande adopting Atterberg limits for classification of soils (Casagrande, 1932), Cooling and Smith (1936) using the ring shear apparatus to measure strength, and the cell test (Geuze and Tan Tjong Kie, 1950) and triaxial test being developed (e.g. Taylor, 1939).

Most test techniques currently in use date back to the period between about 1955 and 1985. The practical and long-standing success of many tests developed in that period can be attributed to the work of Professor Alan Bishop of Imperial College, London, who gave an important review of his contributions at the Stockholm Conference in 1981. Table 1 lists the development of apparatus, instrumentation and methods of soil testing, and gives key references. Whilst there have been few developments in terms of laboratory apparatus since about 1985, major improvements have been made in instrumentation and control systems.

Laboratory testing has often been carried out as part of research into the behaviour of remoulded, reconstituted or pluviated soils, sometimes because those carrying out the testing believed that the behaviour of such material would adequately mirror that of natural soils, and sometimes unwittingly because the effects of sampling disturbance were not recognised. This paper is concerned, however, with the determination of geotechnical properties of natural soils (which may have been remoulded, partly destructured, or loaded to failure in situ) both for research and practical purposes. Remoulded and reconstituted materials are not of primary interest, but nevertheless may be tested, for example, to obtain index properties or reference behaviour (Burland, 1990).

The laboratory testing of natural soils presents many challenges, for example:

- The real behaviour of the soil cannot be determined in the laboratory unless the effects of sampling and subsequent disturbance can be reduced to acceptable levels;
- Stiffness needs to be assessed at stress and strain levels, and along strain paths relevant to the problem being studied;
- Available strength also depends upon state, destructuring and loading path;
- Natural soils are spatially variable, in terms both of fabric (in the sense of Rowe (1972)), and structure (Cotecchia and Chandler, 2000, Barton, 1993), and this variability strongly affects the results of tests carried out on them;

Table 1. Development of Some Soil Laboratory Test Apparatus, Instrumentation and Test Techniques.

Item	Reference
Strength testing	Coulomb (1776)
Direct shear test	Collin (1846), Bishop (1948)
Oedometer apparatus	Terzaghi (1919), Casagrande (1936), Bishop (1982)
Suction plate apparatus	Schofield (1935), Croney et al. (1952)
Compaction test	Proctor (1933), Soil Mechanics for Road Engineers (1952)
California Bearing Ratio	Porter (1938)
Dutch Cell Test	Geuze and Tan (1950), van Duren and Heijnen, (1979)
Ring shear apparatus	Cooling and Smith (1936), Bishop et al. (1971), Bromhead (1979)
Triaxial cell	Taylor (1948)
Cyclic triaxial cell	Grainger and Lister (1962)
Stress path cell	Bishop and Wesley (1975)
Plane strain cell	Cornforth (1964)
Biaxial apparatus	Hambly (1969a)
True triaxial apparatus	Hambly (1969b), Ko and Scott (1967), Sture and Desai (1979)
Directional shear cell	Arthur et al. (1977), Sture et al. (1985)
Hollow cylinder apparatus	Kirkpatrick, (1957), Saada and Baah (1967), Hight et al. (1983)
Resonant column apparatus	Hardin and Drnevich (1972), Drnevich et al. (1978), Drnevich (1985), Stokoe and Lodde (1978)
Torsional resonant column	Pradhan et al. (1988)
Flexural resonant column	Cascante et al. (1998)
Undrained triaxial test	Bishop and Eldin (1951)
Effective stress triaxial testing	Bishop and Henkel (1957)
Stress path test	Lambe (1964)
Residual strength testing	Skempton and Petley (1967), Bromhead (1979)
Constant rate of strain consolidation test	Crawford (1964), Smith and Wahls (1969), Wissa et al. (1971)
Controlled gradient consolidation test	Lowe et al. (1969)
Restricted flow oedometer test	Sills et al. (1986)
Filter paper suction test	Gardner (1937), Chandler and Gutierrez (1986), ASTM (1993)
Lubricated end platens	Rowe and Barden (1964)
Null indicator	Bishop and Henkel (1957)
Pore pressure transducer	Whitman et al. (1961)
Mid-plane pore pressure probe	Taylor (1948), Sodha (1974), Hight (1982)
Local strain gauges	Brown and Snaith (1974), Jardine et al. (1984), Clayton and Khatrush (1986)
Bender elements	Shirley and Hampton (1977), Schultheiss (1981), Dyvik and Madshus (1985), Pennington et al. (1997)
Suction probe	Ridley and Burland (1993)

- Natural soils may be unsaturated, and the engineer must decide how to respond to this;
- Many natural soils have gas or solutes dissolved in their pore water, and any testing system needs to take this into account.

2 OBJECTIVES OF GEOTECHNICAL TESTING

Whether for research or for practice, soil testing needs to fit logically within a general scheme of investigation, and to satisfy a need. Geotechnical investigations are carried out using a wide range of techniques that typically involve boring, drilling, in-situ testing, sampling and laboratory testing. Each has different potential benefits and challenges, as will be discussed later in this paper. To make matters difficult for the inexperienced engineer trying to pick appropriate methods to use, each has strengths and weaknesses that can vary with soil type.

In professional practice geotechnical investigations aim to provide all the necessary data for geotechnical design. The following steps are generally taken:

1. Simplify the ground by dividing it into a series of soil types each of which is expected to have broadly similar engineering behaviour (a process here termed 'soil grouping', or when linked to a particular application (for example road design) 'soil classification');

2. Determine the three-dimensional physical extent (subsoil geometry) of each type of soil, for example using profiling or sectioning, which may employ laboratory ‘index testing’ techniques, and
3. Obtain engineering parameters for the range of problems being investigated, from representative locations (for in-situ tests) or representative samples (for laboratory tests).

Thus in commercial practice geotechnical testing makes its contribution alongside the many other techniques available to the geotechnical engineer. ‘Commercial laboratory testing’ is typically specified according to accepted national or international standards, and in the UK is often quality assured. However it must deliver results that are relevant to the civil engineering development concerned in a timely and cost-effective manner. Standardisation and quality assurance may not always ensure this.

In research, geotechnical testing has a different and arguably unique position, in that it can allow properties of soils to be determined under controlled loading paths, at unique levels of stress and strain, with controlled boundary conditions. It should therefore be no surprise that, despite its inherent difficulties, laboratory testing has made a major contribution over the past decades to our fundamental understanding of the behaviour of natural soils.

The purposes of geotechnical testing can therefore be summarised under two headings:

1. To provide good quality geotechnical design data for
 - Soil grouping;
 - Profiling;
 - Parameter determination.
2. To produce accurate and repeatable data on the behaviour of soils and weak rocks under controlled and relevant test conditions in order to gain new understandings of its fundamental behaviour.

3 STRATEGY AND TACTICS

In order to achieve the objectives discussed in the previous section the geotechnical engineer, whether a practitioner or researcher, must develop strategies to deal with a number of key issues. The strategic issues to be addressed include:

- Making a rational choice between laboratory testing and other methods of obtaining the required information, e.g. for profiling and parameter determination;
- Recognising and dealing with heterogeneity;
- Selecting laboratory testing methods that are as simple, fast and cheap as possible;
- Obtaining representative samples of suitable quality;
- Matching the sophistication of laboratory test methods to sampling quality and end-user (e.g. engineering computation) needs;
- Identifying satisfactory detailed laboratory procedures for the test methods selected;
- Ensuring that skill and knowledge requirements are met;
- Checking that apparatus and instrumentation are functioning at the level necessary to yield worthwhile and meaningful results
- Ensuring that the test results are reasonable.

Sampling and detailed laboratory test methods will be considered in the following sections of this paper.

3.1 *Choosing between laboratory testing and other methods*

A number of authors have considered the advantages and disadvantages of laboratory testing in relation to geotechnical in-situ testing (e.g. Wroth, 1984). None appear to have considered the issue in relation to other possible methods of achieving the objectives that have been identified above, for example, using information from databases and case histories, or avoiding the issue completely by adopting a conservative approach (perhaps, for example, based upon visual description and critical state soil mechanics, or cone testing in conjunction with precedent practice).

Table 2 lists some of the advantages and disadvantages that have been associated with laboratory testing in comparison with in-situ testing. In order to characterise natural soils by laboratory testing it is first necessary to obtain a sufficiently undisturbed sample (Rowe, 1971). The specimen prepared from the sample should be internally uniform, or if not uniform, should be large enough relative to the scale of non-uniformity so that results can be interpreted as if it were uniform.

Table 2. Advantages and disadvantages of laboratory testing relative to in-situ testing.

Laboratory testing	In-situ testing
<i>Advantages</i>	
– Tests can be carried out in a well-regulated environment – Stress and strain levels can be controlled, as can drainage boundaries and strain rates. – Effective strength testing is relatively straightforward – The effects of stress path and history can be examined – Drained bulk modulus (volumetric compressibility) can be determined.	– Test results can be made available during the course of the investigation, and more quickly than laboratory test results. – Appropriate methods (e.g. seismic testing for very small strain stiffness) may be able to test large volumes of ground, ensuring that the effects of large particles and discontinuities are fully represented. – Estimates of horizontal in-situ stress can be obtained.
<i>Disadvantages</i>	
– Laboratory test results will be unreliable whenever samples of sufficient quality or size are unobtainable, for example in most granular soil, fractured weak rocks, and stony clays. For purposes of interpretation, specimens are assumed to be homogeneous. – Tests for strength and compressibility will not produce reliable results if the samples cannot be obtained (e.g. in granular soils) or where disturbance is high. – Laboratory test results (and particularly the results from advanced testing) will only be available some considerable time after the completion of ground investigation field work.	– Drainage boundaries and conditions are not controlled (see Wroth, 1984, for example). It cannot definitively be known whether loading tests are fully undrained. – Tests cannot be made to determine the drained behaviour (e.g. effective strength parameters) of clays, because of the expense and inconvenience of a long test period. – Pore pressures cannot be measured in the volume of soil under test, so effective stresses in slower draining materials remain unknown. – Stress paths and strain levels are often poorly controlled.

However, it must now be recognised that in reality it is impossible to test truly undisturbed specimens. The effects of disturbance range from loss of (normally anisotropic) total stresses (when the highest quality samplers and testing methods are used in clays) through to a complete inability to recover tube or core samples (as is typically the case with alluvial sands and gravels, when traditionally laboratory work is very restricted and penetration testing is normally used for ground characterisation). In this context, in the absence of methods of obtaining undisturbed specimens, it seems reasonable to test reconstituted or pluviated specimens, provided that the results are treated with great caution. The application of laboratory test data (even if obtained from specimens of sufficient size) in the prediction of the behaviour of much larger volumes of inhomogeneous natural ground is the subject of recent and current research (for an example, see Hicks and Samy, 2002).

3.2 *Recognising heterogeneity – imaging, particle measurement, visual description, and profiling*

Heterogeneity occurs in soils and weak rocks at all scales, and may vary in a number of distinct ways, for example as a result of:

- Differences in the arrangement of particles of the same size (termed ‘structure’ by Rowe (1972) – for example dispersed and flocculated structures in clays.
- Preferential orientation of particles, as a result of depositional or stress-induced anisotropy.
- Differences in particle size and shape as a result of variations in the depositional environment.
- The arrangement of particles of different sizes (termed ‘fabric’ by Rowe). – for example, laminated and varved deposits, fissure fabric.
- The morphology of inter-particle contacts, as for example in locked sands.
- The morphology, amount and composition of cementing (part of ‘bonding fabric’).
- The presence, orientation, spacing and persistence of discontinuities, such as fissures, joints, and faults, together with variations in their surface textures.
- Variations arising due to physical and chemical weathering.

Additional heterogeneity is then introduced as a result of sampling and testing techniques.

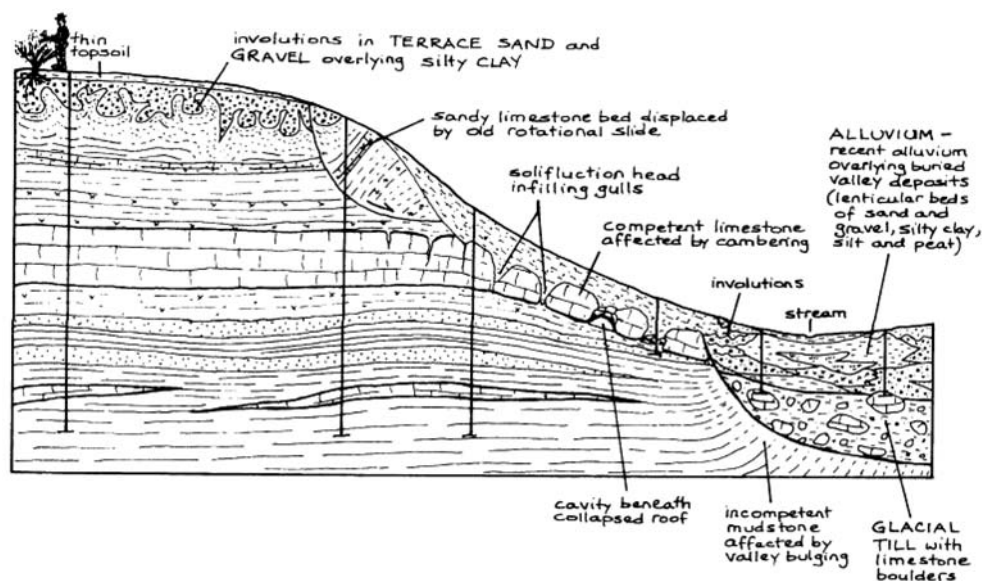


Figure 1. Geometric variability of the ground – a ground model from Fookes (1997).

As can be seen, the characteristics of a sediment vary to a great extent as a result of its depositional environment, but can be increased by both post depositional and late stage processes such as cementing and weathering. Heterogeneity also occurs at every scale, but from an engineering point of view tends to be reduced as a result of gravitational compaction and aging.

At the macro scale, Fookes (1997) has provided an excellent description of ground models (which he terms 'geological models') (Figure 1) that may occur in different depositional and weathering environments, and these examples should act as a warning to engineers who might otherwise believe that subsoil geometry can be interpreted solely from a series of widely-spaced vertical boreholes or probes which can only ever encounter a minute proportion of the subsoil that will be affected by construction (Broms, 1980). The determination of a satisfactory ground model for a site (alongside a suitable contamination model where appropriate) is an essential pre-requisite for the interpretation of necessarily limited classification and profiling data, and for the selection of representative samples for more sophisticated laboratory testing. This requires a detailed knowledge of site topography, interpretation of topographic features in terms of the underlying ground (for example using remote sensing), and firmly based on knowledge of the depositional geology, geomorphology and regional geological setting of the site.

Recognition and recording of heterogeneity at the large scale (for example through geological interpretation, seismic sectioning, probing and penetration testing, borehole data, visual description of exploratory pits and shafts, and in-situ testing) is beyond the scope of this paper, although it provides the vital context within which to choose samples for testing, and to assess how representative data from tests are likely to be of the mass of ground affected by construction. In most ground, however, boundaries between different soils are gradational, and properties within soil types are not uniform, spatially or directionally. Examination in situ, in addition to providing detailed profiles, is normally the only satisfactory way to fully understand the ground, but in particular to record discontinuity patterns, assess larger scale variations in composition and grain size, and the presence of coarse elements such as boulders, chert and flint.

From a laboratory testing perspective it would be ideal if the ground could be divided into a few layers or three-dimensional zones, each of which could be considered internally uniform and isotropic, and therefore could be characterised by a limited number of laboratory tests. This is not the reality.

If heterogeneity is to be assessed in the laboratory, and simplified ground profiles developed, a great many determinations of each selected property (perhaps, for example, moisture content) will be required. Because of this, profiling is most effectively carried out using quick, simple, relatively cheap, techniques, but they must be shown to provide repeatable results. For many years the standard practice for high-quality UK commercial ground investigations was to carry out large numbers of moisture content and Atterberg limit tests, but in more recent years this has generally fallen out of favour with practitioners, largely because of the cost of testing and the time for its completion, but also because of a perception of falling

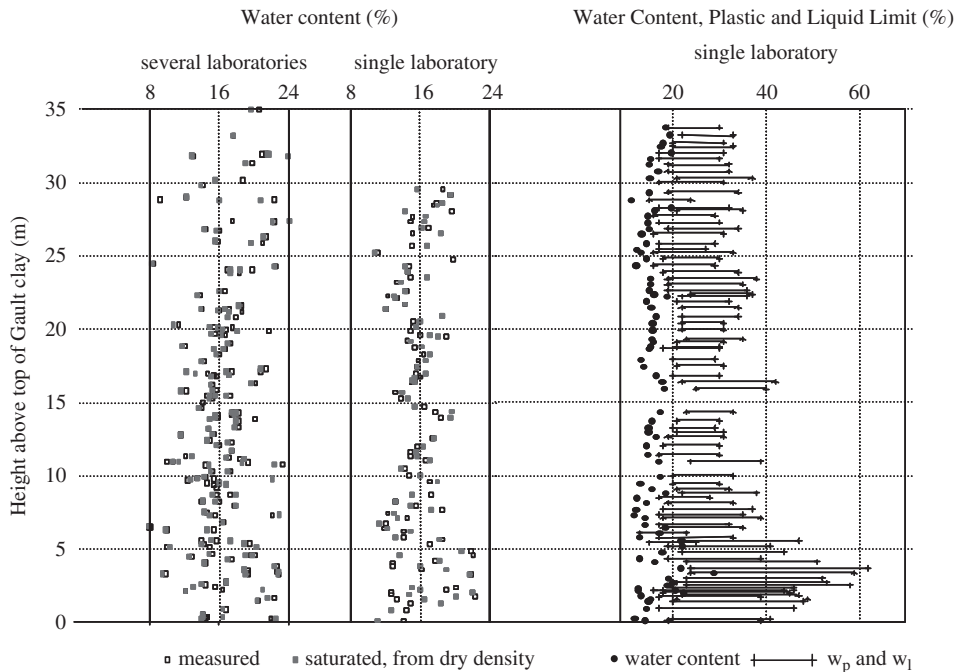


Figure 2. Water content and plasticity data from the Cenomanian Chalk.

levels of test reliability, and in particular repeatability. Figure 2 shows profiles of water contents and Atterberg limits at a site in the Cenomanian Chalk of south-east England. The left-hand graph shows moisture contents determined by a number of laboratories. Next to it are results obtained by two technicians in a single laboratory. On the right, these are compared with Atterberg limits from the same laboratory, using the 4-point cone method. Despite the sometimes apparent uniformity of its appearance, this deposit has rhythmicity in its stratification, reflecting climatic fluctuations which affected both organic productivity in the sea and clastic input (Clayton et al., 2002). Routine laboratory testing, with fewer determinations carried out by a number of laboratories and technicians failed to identify this, and indeed produced such a large scatter in moisture content and saturated moisture content values that the performance was significantly underestimated. To investigate the full variability even closer testing intervals would be required.

At the specimen level, heterogeneity can be recognised by careful observation, imaging and measurement. Visual description is the normal starting point, best carried out by splitting (rather than cutting, which smears the soil and conceals particle fabric) undisturbed samples longitudinally, photographing them (on a suitable background with a standard length and colour scale) when wet, and as they dry. Fissures and bedding discontinuities generally show up well, and can be recorded. Coarser layers and lenses dry first, emphasizing fabric. Both radiographs and X-ray densimetry (Paul et al., 1996) can contribute to an assessment of heterogeneity. At the smaller scale, optical and scanning electron microscopy can be used to look for evidence of spatial particle size variations, anisotropy, particle flats and inter-penetration, and bonding.

3.3 Dealing with heterogeneity – classification, grouping, specimen size and test orientation

There is little point in carrying out laboratory element tests in highly heterogeneous sediment unless there exists some strategy for sampling and testing, and also for the rational use of the data that will be produced. From the laboratory testing perspective the essential steps would seem to be:

1. Recognition and assessment of heterogeneity (see above).
2. Classification testing to provide numerical data.
3. Soil grouping, to recognise zones containing materials expected to have broadly similar properties.
4. Identification of locations of materials for laboratory testing.
5. Determination of suitable specimen size and test orientation.

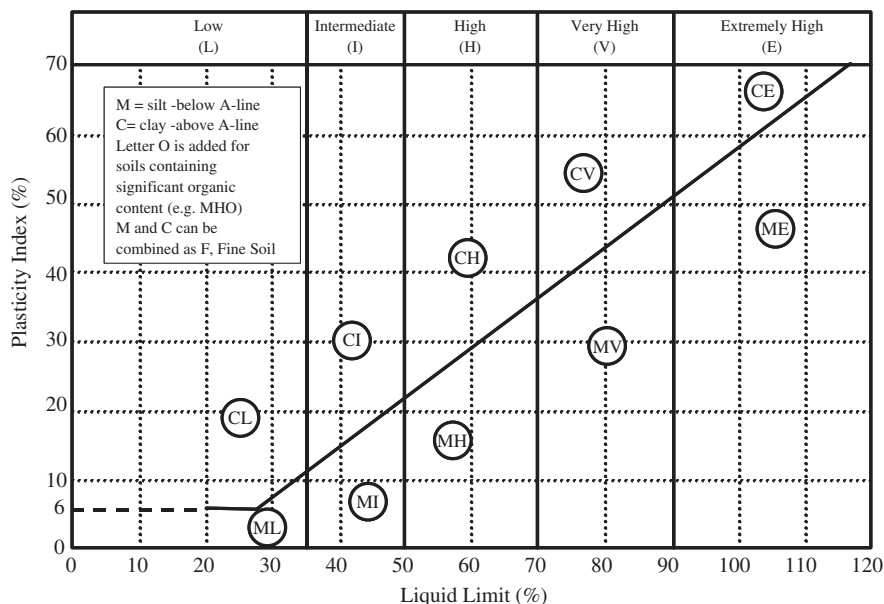


Figure 3. Plasticity chart for the classification of fine soils (after BS5930:1999).

Begemann recognised the importance of heterogeneity, and understood that in the complex finely-layered alluvial soft clay and sand of the Netherlands it was more important to determine the proportion of (relatively strong and incompressible) sand and (relatively weak and compressible) clay in the soil profile than to attempt to determine precise compressibility parameters for each layer. As a result he developed the continuous Delft stockinette sampler (Begemann, 1961), which allows an appropriate selection of test samples to be made in the context of a good knowledge of the variability of the ground. In appropriate ground conditions the CPT friction cone and piezocone can now provide such data almost as effectively, but at much lower cost, although the absence of a sample prevents visual assessment and subsequent laboratory testing.

Visual observation, continuous sampling or continuous in-situ testing would seem to provide the greatest number of options both in terms of obtaining classification data and selecting the locations of specimens for preliminary testing. But, due to lack of experience, unsuitable ground conditions and financial constraints this is rarely done. In most commercial practice samples intended for testing must be taken at the same time that samples are acquired for classification purposes. But in research is it feasible to carry out two stages of sampling, with the second stage targeting the acquisition of high-quality samples for the determination of soil parameters.

The two test methods routinely used as the basis for laboratory classification of coarse soils are wet sieving (for particles coarser than about 63μ to 75μ) and sedimentation (suitable for spherical quartz particles between about 60μ and about 2μ). For non-spherical particles these two tests do not measure the same parameter. Because there is little evidence to support the notion that particle size has a dominant effect on the geomechanical behaviour of coarse soils, it has been common practice to use mean particle size (by weight) as the means of identifying layers of different coarse soil.

The basis for *classifying* fine soils in the UK and the US is the system proposed by Casagrande in 1947, using the Atterberg limits (Liquid Limit and Plasticity Index) (Figure 3). Other parts of the world, with different ground conditions, have not always adopted this approach. For example in South Africa the practice is to use the Jennings, Brink and Williams system (Jennings et al., 1973). This provides a means of producing a soil profile on the basis of the origin of the materials that are encountered.

Soil grouping is currently carried out in a number of ways. In countries such as the USA and the UK, where there has been a strong tradition of laboratory testing, routine soil grouping (e.g. for building construction) has been done on the basis of particle sizing (for coarse soils), and moisture content and plasticity (for fine soils), supported by soil description. Elsewhere the preferred techniques range from in-situ cone penetration testing (e.g. in the Netherlands) to visual soil description alone (as in South Africa).

3.4 *Selecting appropriate laboratory testing apparatus and method*

In an earlier section of this paper we discussed the purposes of geotechnical testing, both for practice and research. It is, perhaps, obvious that laboratory tests may not be 'fit for purpose' for a number of reasons including, for example, poorly calibrated transducers, tests carried out on badly disturbed specimens, etc. However, before coming to this level of detail it is appropriate to consider higher-level factors that may reduce the value of test results, or in extreme circumstances make them worthless. Some examples are given below.

One of the most common reasons for routine laboratory soil testing is to obtain parameters for geotechnical computation. The most basic decision that an engineer commissioning or carrying out testing on clays must make is whether short-term (undrained, $\phi_u = 0$) analysis or long-term (effective stress) analyses are required (Bishop and Bjerrum, 1960). Clearly effective-stress triaxial testing is of little use in the routine design of pad foundations on clay, but is essential in the design of slopes in heavily overconsolidated clays.

Whether implicitly or explicitly, laboratory testing is carried out with some framework or constitutive model of soil or rock behaviour in mind. Increasingly use is made of finite element or finite difference analyses, and to use these tools the analyst must select an appropriate constitutive model for the soil. If the analyst wishes to use a critical state soil mechanics approach then a very different set of tests will be required than if it is proposed to use anisotropic elasticity coupled with Mohr-Coulomb plasticity. Failure to recognise the geotechnical framework within which testing is carried out can lead to the production of almost worthless test results. This framework defines the parameters and measurements to be made, and therefore the appropriate apparatus and instrumentation to use.

3.5 *Obtaining representative samples of suitable quality*

Hvorslev (1949) concluded that it is necessary, if laboratory test results are to be of use, for specimens (and therefore samples) to be representative of all the features of the ground from which they are taken. Effectively this means that test specimens made up from samples must be sufficiently large to contain a representative volume of material, so that:

- Specimens contain all the particle sizes that are to be found in-situ, and in the same proportions;
- The maximum particle size is not large in comparison to the minimum specimen dimension, to avoid boundary effects;
- The specimen contains sufficient discontinuities (joints or fissures) to be representative of the mass from which it is taken;
- Fabric (the arrangement of different particle sizes, for example as varves, silt dusting on fissures (termed 'fissure fabric'), etc.) is fully represented.

Clearly, given that heterogeneity can occur at all scales, laboratory testing may not always be the most effective way to obtain data. For example, the compressibility of a near-surface rock mass will be controlled by the stiffness, frequency and orientation of the discontinuities it contains. Discontinuity spacing is wide compared with available specimen sizes, but in any case no method exists to remove a representative block from the ground without causing disturbance to the block contacts. Field observation and testing are required.

The factors described above not only control the minimum size that a specimen must be if it is to provide results that reflect the properties of the ground from which it is taken, but also (in conjunction with the application of the results) define appropriate loading and drainage directions. These issues are considered in more detail in the sections below on test technique.

In addition, if specimens of natural clays are to reflect correctly the mechanical behaviour of the ground from which they are taken, then mechanical disturbance must be kept to low levels. Disturbance can take place during boring, during sampling, during transporting and storage, and during the early stages of laboratory testing itself. These issues are considered in the sections below on sampling disturbance.

3.6 *Optimising the test technique*

Once appropriate apparatus and a general test method have been defined, it is necessary to identify those other factors and test details that can affect the outcome of a test. These include:

- Availability of the required apparatus.
- The availability and commitment of appropriately skilled people to carry out the tests.

- Familiarity with the apparatus and instrumentation.
- Availability of adequate calibration, logging and if required, control systems.
- Experience of preparing and testing specimens of the expected soil types in the apparatus to be used.
- Knowledge of suitable detailed test methods.
- Supervision and review of results.

In the authors' opinion, the best results come when the testing and associated systems are kept as simple as possible whilst still meeting the objectives. It is essential, unless sufficient time is available for test development and repeat testing (as is normally necessary with research testing) to make sure that what is proposed does not exceed the capability and experience of those involved.

Measurement systems and test methods are considered in two sections below.

3.7 *Checking the results of tests carried out by others*

There can be no doubt, based upon the authors' own observations, that there is a great deal of inappropriate or poor quality soil testing currently being carried out across the world. It is therefore necessary to be vigilant, and to carry out a number of checks before accepting and using laboratory data produced by a third party.

The following tactics are helpful when relying (as most of us will have to) on others to carry out our tests:

1. Take an interest in your testing. For commercial testing, read national and international standards and try to understand why your particular test specification demands the things it does;
2. Visit the laboratory as frequently as possible during testing, and discuss emerging test results with the researcher, or for commercial testing with the laboratory manager and his technicians;
3. Always ask for copies of instrument calibration data, raw test data, and spreadsheets used to compute the results from the raw data;
4. Carry out spot checks by asking to see example technician worksheets from which the final results given to you have been computed;
5. Compare your test results with those in similar ground conditions, published in technical papers, in journals and conferences such as the International Workshop on Characterisation and Engineering Properties of Natural Soils;
6. Consider carrying out repeat testing using different researchers or in commercial work, different laboratories;
7. Compare individual test results with others of the same type on similar soil (e.g. by plotting profiles of test data);
8. Compare parameters obtained from your test data with values obtained from other (for example in situ) techniques.

4 PRINCIPAL TEST TYPES AND LABORATORY HARDWARE

This section considers the principles, test methods, limitations, necessary calibration and corrections (e.g. filter drains, membrane, load) for the range of soil testing currently available. It divides test types and equipment under the following heads:

- Classification, index, compaction and CBR testing;
- Consolidation and compressibility testing;
- Shear testing;
- Triaxial testing;
- Advanced loading systems;
- Permeability testing.

and considers the purpose for which the equipment was developed, and the form currently taken by 'standard' apparatus. Shortcomings are noted, where these have been recorded.

4.1 *Classification, Index, Compaction and CBR Testing*

This group of tests is taken to include:

- Moisture content determination;
- Particle size and shape;

- Atterberg limit tests;
- Specific gravity (particle density) determination;
- Compaction and CBR testing for highways;
- Basic strength tests (e.g. unconfined compression test, undrained triaxial, Brazilian tensile test, point load test, etc.)

With only one or two exceptions, the results of these tests are used empirically, or in comparison with each other. The main requirement is that the results should be reproducible, at least to the level required for practical engineering, when carried out by different researchers or technicians in the same laboratory, and in different laboratories. The test apparatus is generally simple, and since it is well known, will not be discussed in detail here. Despite the simplicity of the apparatus there is ample evidence that in practice it is difficult to achieve acceptable levels of repeatability (see later sections on test methods).

4.1.1 *Water (or moisture) content*

Potentially the simplest of all geotechnical laboratory tests is the determination of water (or moisture) content, which when combined with specific gravity, bulk density and Atterberg limits provides the basis for assessing the state of soils. BS1377:1990 Part 2 states that moisture content is ‘the amount of water, expressed as a proportion by mass of the dry solid particles’. It defines a soil as ‘dry’ when ‘no further water can be removed at a temperature not exceeding 110°C.’ The apparatus required to carry out moisture content tests is well described in national and international standards. For example, BS1377:1990 requires an oven (but not a microwave oven) capable of holding its temperature between 105°C and 110°C, containers with lids, balances with suitable accuracies and ranges, and a vacuum desiccator. The importance of these components will be discussed later.

4.1.2 *Particle size and shape*

Advances in computing, and in particular discrete element modelling, suggest that it may eventually become possible to predict soil behaviour from the particle upwards. There is some evidence that particle characteristics such as size and shape may then provide the basis for a new, more fundamental, soil mechanics (Gilboy, 1928, Olson and Mesri, 1970, Santamarina and Cho, 2004, Clayton and Obula Reddy, 2006). At present, however, particle characteristics are often used in much the same way as colour may be, to provide a basis only for distinguishing different types of deposit. Even when attempting to determine permeability, the presence of fabric and the dominant effect of fines make mean particle size an unreliable guide to field permeability.

There are now many methods of determining particle size, for example by:

- sieving
- sedimentation
- optical sizing
- electrical sensing
- laser diffraction
- image analysis, etc.

Different test apparatus yield different particle size distributions, and are applicable only to limited differing ranges of particle size. Soil testing laboratories have traditionally used, and continue to use sieving for coarse soil and sedimentation for fine soil.

4.1.3 *Atterberg limit (plasticity) tests*

The Liquid Limit is most commonly determined using the Casagrande cup (Casagrande, 1958) (Figure 4a). The apparatus is simple, but suffers from some problems that can lead to a lack of reproducibility. For example, the properties of the base can lead to significant differences in result, and the hardness should be specified and checked (for example, South African highways code TMH1(1979) requires a Shore D hardness value of 85–95 at $23 \pm 2^\circ\text{C}$). BS1377 states that the base must be made of “vulcanized rubber of hardness between 84° and 94° IRHD (International Rubber Hardness Scale D) and resilience between 20% and 35% at $20 \pm 2^\circ\text{C}$ determined in accordance with BS 903”. Hardness may vary as resilience decreases with the age of the apparatus, and needs regular checking. BS1377:1990 therefore requires hardness to be checked every two years using the Lupke pendulum apparatus. At least two types of grooving tool appear to be in use across the world – the impact of these on test results is not known to the authors.

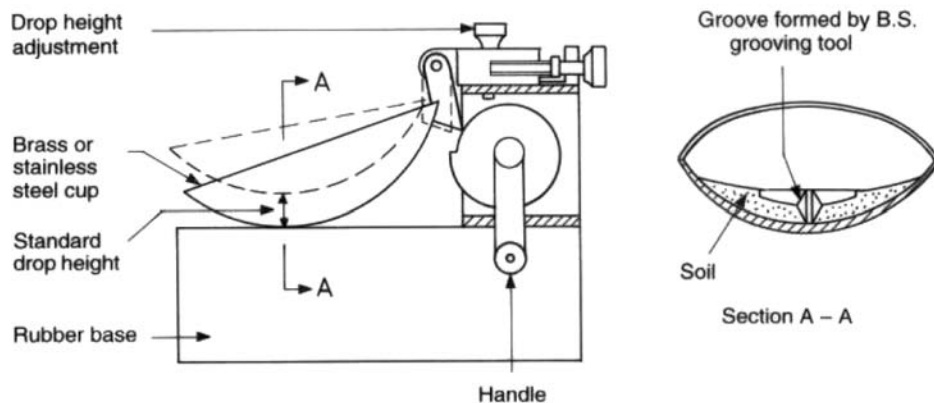


Figure 4(a). Casagrande Liquid Limit Apparatus.

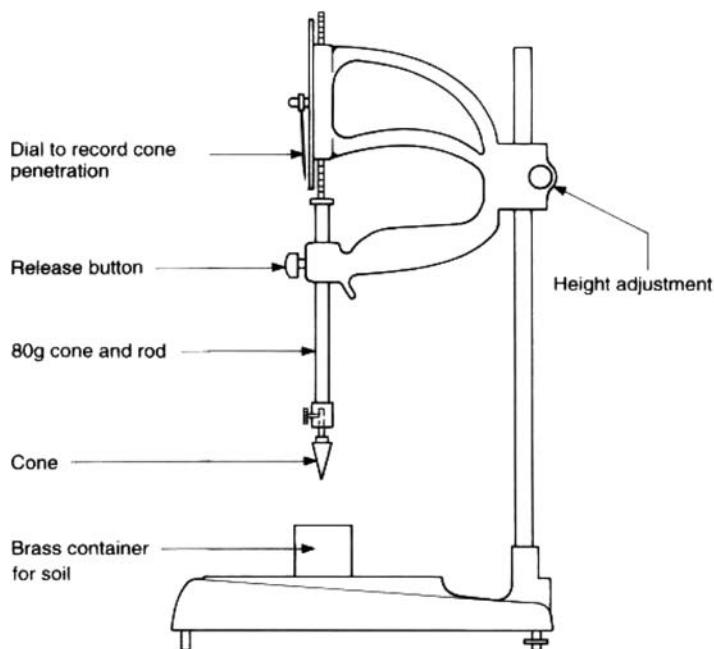


Figure 4(b). Apparatus for BS1377:1990 80 g fall cone Liquid Limit test.

Results obtained from the Casagrande cup are known to be variable and operator dependent (see later), and for these reasons the preferred (“definitive”) method in the British Standard uses an 80 g 60° fall cone to determine the Liquid Limit (Figure 4b).

4.1.4 Compaction and California Bearing Ratio (CBR) testing

Compaction tests are carried out to a range of standards and using a variety of apparatus. Most national standards appear to use tests that approximate to the original Proctor compaction test, which tested 1/30th of a cubic foot (approximately 1 litre) of soil in a 4” (approx. 100 mm) diameter mould, compacting the soil in three layers, each receiving 25 blows from a 5.5lb (2.5 kg) 2 inch (50 mm) diameter rammer dropping through a height of 12 inches.

Table 3. Effect of de-airing specific gravity test specimens (Sherwood, 1970)

Soil type	Road research laboratory results			Other test houses		
	Mean	Min	Max	Mean	Min	Max
Bagshot beds	2.70	2.69	2.72	2.66	2.54	2.81
Gault clay	2.75	2.74	2.77	2.70	2.58	2.84
Weald clay	2.79	2.78	2.81	2.71	2.60	2.85

Over the years other amounts of soil and methods of compaction (e.g. the Modified AASHO (BS 4.5 kg) method and the BS Vibrating Hammer method) have been added in a number of national standards. The BS vibrating hammer method uses a CBR mould, with a volume of about 2.3 litre. Fundamentally there is a dilemma; the need to restrict the volumes of soil required for compaction tests to practical amounts itself reduces the validity and repeatability of the results. As it is, BS1377:1990 requires 40 kg of soil when the CBR mould is used, if that soil is considered susceptible to crushing. Samples of this size are rarely taken.

Hand-held hammers are most commonly used for compaction tests, despite the fact that a range of automated compactors (which give a reproducible pattern of blows, and should therefore lead to more repeatable test results) has been available for many years. This is not only due to the high cost of these units. The authors' experience has been that they are sometimes unwieldy in use, and unreliable.

CBR testing is carried out both in the lab and in the field. The test was originally developed by the California State Highway Department for the evaluation of the strengths of sub-grades under pavements of known performance. The data collected subsequently led to an empirical method of pavement design. The loads necessary to achieve given 2.5 mm and 5 mm penetrations of a 50 mm (approx.) diameter plunger are compared with those from a standard crushed stone. Because of the plunger diameter the test is not suitable for soils containing coarse particles. BS1377 sets the limit at 20 mm. This is an empirical test, and clearly its results will depend on such details. Care also needs to be exercised in the use of the results. For example, an in-situ test on a freshly excavated over-consolidated clay will not give results that are useful for pavement design, since in the long term such soil will swell under the relatively low effective stress levels under a pavement, losing most of its stiffness and strength.

4.1.5 *Specific gravity (particle density) determination*

The determination of fundamental quantities such as void ratio, specific volume and air voids content require knowledge of specific gravity (sometimes termed 'particle density'). Over the years, standard methods have made use of the pycnometer for coarse-grained soils such as gravels, and the density bottle for finer soils, using vacuum to encourage the removal of air. However Krawczyk (1969) found that results using these methods were consistently too low, as a result of the difficulty of fully extracting air from the containers. He showed that end-over-end shaking of a 1-litre gas jar produced better and more reproducible results. This method (although not widely used in the authors' experience) provides much improved results and is now recommended by the British Standard (BS1377:1990, part 2, clause 8.2). Table 3 shows results quoted by Sherwood (1970) for three clays tested with special attention to de-airing by eight UK Road Research Laboratory operators, compared with values obtained on the same soil by 30 other test houses. Not only is there considerable variability when using traditional methods, but the mean is also greatly affected. Effective de-airing increases particle density by between 0.04 and 0.08. Tanaka (2002) has reported similar discrepancies between laboratories measuring specific gravity.

4.1.6 *Basic strength tests*

A number of basic strength tests have been used in the testing of soils, such as the:

- Unconfined compressive strength test (UCT).
- Unconsolidated undrained (UU) triaxial compression test.

and for rocks

- Point load test.
- Brazilian tensile test.

In the UK the unconfined compressive strength test has largely fallen out of use for soils. The Point Load and Brazilian Tensile Tests are not particularly well suited to the testing of weak rocks, because the scatter in measured values is large compared to the mean.

In UK commercial testing, use of the unconsolidated undrained triaxial compression test is widespread. The results are used as if they are representative of the in-situ undrained strength of the material, despite the fact that research has shown that both:

- sample size (for example, Bishop and Little, 1967, Agarwal, 1968, and Marsland, 1972), and
- sampling disturbance (for example, Seko and Tobe, 1977, Hight, 1986, Chandler et al., 1992).

have a very large effect on the results. This state of affairs seems to prevail in other countries, and suggests that measured undrained strengths are not dangerously high (i.e. more than compensating factors of safety used in design). Alternatively the findings of Hight (1986) and Chandler et al.(1992), that undrained shear in stiff clays may be overestimated when tube sampled as a result of effective strength changes, is either unusual, or is compensated for by the effects of destructuring (Hight and Jardine, 1993). The determination of strength will be discussed in a later section.

4.2 Consolidation and compressibility testing

Geotechnical testing is routinely carried out in order to determine the compressibility or stiffness of soil, and/or to obtain some measure of the rate at which compression will take place. The types of apparatus most commonly used for these purposes are the oedometer (Terzaghi, 1923, Casagrande, 1936, Bishop, 1982), the hydraulic consolidation cell (Rowe and Barden, 1966), and triaxial apparatus (Bishop and Henkel, 1957, 1962).

Laboratory measurements of compressibility using routine techniques are in many soils unlikely to be representative of field values. Compressibility may be significantly overestimated, both in soft clays (due to sample disturbance) and in stiff and hard soils (due to bedding effects). In weak rock the lack of representative fracturing in the specimen may more than compensate for bedding effects. In almost all materials tests on small specimens will, because they are not representative of fabric, very significantly underestimate the speed of dissipation of excess pore pressures. If they are critical to a design, laboratory determinations of Coefficient of Consolidation should be checked against the results of field testing, perhaps determining C_v from a combination of field in-situ permeability testing and laboratory determinations of Coefficient of Compressibility, m_v , through the equation

$$C_v = \frac{k}{m_v \gamma_w}$$

4.2.1 The oedometer test

The standard oedometer, developed from apparatus originally used by Terzaghi to determine the permeability of clay soils (Clayton and Müller-Steinhagen, 1995), is thought to have been redesigned by both Casagrande (1936) and Bishop (1982). The specimen is typically 3inch (76 mm) diameter, and 3/4inch (19 mm) high, and is immersed in a water bath, to ensure that the pore pressure in the specimen is controlled, and to avoid evaporation during the relatively long test period. Vertical loading is applied by weights on a hanger/lever-arm system. The Bishop back-loading system allows re-setting of the loading arm position during testing, which is an advantage when testing more compressible soils. The more commonly used Casagrande front-loaded design is easier to use as access is only needed to one side of the machine. The need for weights (with their attendant health and safety implications) has been avoided by more recent systems based on compressed air loading and electronic load/displacement measurement.

In a conventional commercial test between five and ten loading or unloading stages are applied, each vertical stress level being approximately twice (or one half during unloading) of the previous one. A dial gauge or displacement transducer mounted on the top of the loading yoke records the compression of the specimen during each stage. Loads are normally held for a period of 24 hours.

In soft clay the oedometer has been routinely used to determine the 'preconsolidation pressure' (p_c) (Casagrande, 1936, Schmertmann, 1953) and, from the slope of the virgin compression curve, the compression index, C_c , or critical state parameter, λ . In stiffer material, for example overconsolidated clay, the coefficient of compressibility, m_v , which varies as a function of the stress range over which it is determined, is found in place of the compression index. The apparatus potentially

suffers from a number of inadequacies, resulting from the apparatus and the way in which it is used:

- The small thickness of the specimen;
- Apparatus compliance, and compression of any filter papers used at the specimen ends;
- Bedding between the specimen and porous stones;
- The small volume tested;
- Vertical drainage;
- Incremental loading;
- The applied stress path during loading;
- Sampling disturbance and poor storage;
- Immersion of the specimen in water.

The small thickness of the specimen is necessary in order to avoid significant shear stress between the rigid side walls and the soil as compression takes place. The use of a higher aspect ratio would mean that the applied vertical stress would not be converted to a uniform vertical total stress, and that significantly lower stress levels would be applied at the base of the specimen.

The small specimen thickness has a particularly important effect in stiffer materials (stiff clays and weak rocks) where bedding effects can predominate, and stiffness (i.e. constrained modulus) can easily be underestimated by an order of magnitude (Chandler and Davis, 1973, Clayton et al., 1991, Hobbs, 1975). Soils with particles larger than about 2 mm (the limit between coarse sand and fine gravel) should not be tested in the oedometer because of the small specimen height. The small height also means that in stiff fissured soils there will be almost no horizontal or sub-horizontal discontinuities in the specimen, and that (in the absence of bedding effects) measured vertical stiffness would be higher than that of the in-situ mass that the test aims to represent.

The small volume tested will also affect the rate of drainage if, as is often the case, fissure fabric (in the sense used by Rowe) is present. Dustings of silt on fissure surfaces provide fast drainage paths in many soils, but much larger specimens must be tested if the effect of these is to be represented in the test results. Rowe (1968, 1972) gives examples which show ratios between back-calculated and oedometer C_v values of up to 10^3 .

Vertical drainage is a problem because most soils have significant permeability anisotropy created by depositional fabric structure, the horizontal permeability being much larger than the vertical. Permeability anisotropy, which occurs in uniform soils because of the deposition of particles on the flattest faces and in other soils because of laminations, can be associated with k_h/k_v ratios of the order of 10^2 to 10^3 . Since horizontal permeability dominates drainage in almost all geotechnical problems, results obtained from oedometer testing are rarely going to be of value.

The stiffnesses and void ratios obtained from oedometer tests depend upon the increments used during loading. Crawford (1964) noted that the rate of strain in an oedometer test may be more than 10^6 times that in the field, and that test procedure has a large influence on estimated preconsolidation pressure in soft clays. Constant rate of loading and constant rate of strain tests (Smith and Wahls, 1969, Aboshi et al., 1970, Wissa et al., 1971) go some way to overcome this problem.

At least in theory, an oedometer specimen is compressed vertically, with no lateral strain occurring. In clay in the field, fast loading of limited width compared with the thickness of the stratum (e.g. from a pad foundation) will lead to an initial change in shape without change in volume, and this will be followed by volumetric strain as dissipation of excess pore pressures occur. Under a wide load the soil will not be able to undergo change in shape until dissipation occurs, when strains will take place with little lateral yield. In the oedometer the specimen does not follow a first-loading K_0 path until post-yield stresses are reached, and in reality the actual effective stress path varies not only with previous loading but also with the degree of fit of the specimen within the oedometer ring. In stiffer materials this is a problem because the amount of vertical strain is dependent on the effective stress path that is applied, which in the oedometer will not be the same as under a foundation in the field (Simons and Som, 1969).

Sampling disturbance and the effects of poor storage have long been known to have a significant effect on oedometer results in soft clays, where they reduce the observed compressibility, and modify the void ratio/logarithm of pressure curve, obscuring the 'preconsolidation' pressure in lightly overconsolidated clays (Schmertmann, 1955, Berre et al., 1969). Because it is likely that empirical methods of correction (e.g. Schmertmann, 1953) are unreliable, it is now considered better practice (in the light of advances in our knowledge in this area) to improve the quality of sampling, storage and specimen preparation.

The effect of submerging a specimen in water, and particularly in distilled water, can be dramatic. Soils having high suctions will swell if left lightly loaded. The apparatus should be loaded without flooding

if the compressibility of such soils is needed. Soils containing soluble salts will also swell, as a result of osmotic effects. Oedometer tests on this type of material demand preliminary analyses of porewater chemistry and the use of a bespoke fluid around the specimen, if this effect is to be avoided.

4.2.2 *The hydraulic oedometer*

The hydraulic oedometer, sometimes known as the Rowe cell, was developed in order to avoid some of the problems discussed above. As with the standard oedometer, the cell has rigid sidewalls. But here the vertical loading is supplied by a water-filled rubber (or more recently, neoprene or similar) membrane, known as a 'rubber jack'. Some modern neoprene jacks seem considerably stiffer than the latex rubber ones that were originally used, and it is possible that this may reduce the vertical stress on the specimen below the applied fluid pressure, particularly in the case of more compressible soils. Cells can be up to 10 inches (250 mm) in diameter, although obtaining undisturbed samples of this size can be a challenge, and involve major expense. The aspect ratio of the specimen is about 1 (vertical) to 4 (horizontal), in order to avoid side friction effects (see above).

Drainage can be either vertical (to sintered bronze discs at the top or top and bottom, of the specimen) or radial (inwards to a central vertical sand drain or permeable filter, or outwards to a permeable plastic sheet installed on the periphery). The vertical strain across the upper surface of the specimen can be made free or fixed, depending upon whether the upper boundary is flexible or rigid. Pore pressure is normally measured at the centre of the base of the specimen, although provision generally exists for it to be measured at other positions. Coefficients of consolidation can therefore be determined from two sets of independent measurements.

This apparatus can be used for the laboratory determination of the horizontal coefficient of consolidation of layered soils, although it may be difficult (as a result of the natural variations in the thickness of faster draining layers) to establish representative values without carrying out a large and expensive suite of tests.

4.2.3 *Triaxial dissipation tests*

The triaxial dissipation test has, in the past, sometimes been used to determine the compressibility characteristics and speed of drainage of cohesive soils. It was classed as a 'standard test' by Bishop and Henkel (1962). The test uses the normal equipment for effective stress triaxial strength testing (but without the need for a loading frame), which is, of course, quite widely available. The test is carried out on specimens with an aspect ratio of either two (for more permeable soils) or one (for most cohesive soils), and with vertical drainage (i.e. without the use of filter paper side drains).

Triaxial dissipation testing has a number of advantages:

- A 100 mm diameter specimen is sufficiently large to include representative fissures and fabric for many soils, and to permit tests to be carried out on soils containing fine gravel particles.
- The measurement of both pore pressure and volume changes allows two independent values of coefficient of consolidation to be determined from the same specimen.

The test can be carried out on partially saturated specimens (for example, embankment dam fill), where initial compression of air voids makes interpretation of C_v from height change measurements (e.g. in the standard oedometer) rather difficult.

4.3 *Direct and simple shear testing*

A number of types of direct and simple shear test apparatus are currently in use. These tend to be applied in specialist fields. For example, the traditional direct shear 'shear box' test and the ring shear test are most frequently used to determine residual strength parameters, the large (300 mm) shear box test is used for tests on reinforced earth fill, simple shear testing is most popular in offshore ground investigations, and the constant normal stiffness (CNS) shear box has been used for rock-socket pile design. Common features of all tests of this type are their mechanical simplicity, which makes them of practical interest, and potential uncertainty regarding the stresses in the soil they test, which can make their interpretation difficult.

4.3.1 *The direct shear test*

The standard Bishop shear box used in the UK for many years has a 60 mm × 60 mm specimen, and the larger box is 300 mm × 300 mm in plan. In the USA and Japan circular boxes are also in use. In the original (and still much used) apparatus normal (vertical) stress is applied via a hanger, lever

arm and weights system. The rate of displacement during shear is constant, and is set using an electrical motor and gearbox. In more modern boxes the vertical load is applied by a pneumatic system, the box is displaced by a stepper motor, and loads and displacements are measured using transducers.

There appear to be few requirements that need to be met in designing a shear box. The specimen width and length should be large compared with the height, in order that shear stress between the soil and the side of the box does not reduce the normal stress on the mid-height section (a ratio of about 3 seems common). Two screws are provided to hold the upper and lower parts of the box together during specimen preparation, and a second pair of screws allow the user to separate the two halves of the box after consolidation and before shearing, so that there is no friction between the two pieces of metal. The box is normally mounted in a water bath, to avoid evaporation and ensure that pore water pressure is known (but see our comments on this in relation to oedometer testing). It is important to ensure the alignment of the applied forces at the split in the box, in order to prevent moments, and as a result non-uniform normal stress.

In the early days of soil mechanics the direct shear apparatus was routinely used to determine the undrained shear strength of clays. By the 1950's the triaxial apparatus became more widely used because of lack of control of drainage in the shear box.

Drainage occurs primarily at the top and bottom of the specimen, where there are porous plates, but also through the gap between the top and bottom halves of the box. Because specimens are free to drain at all times, and there is no pore pressure measurement, undrained tests can only be carried out on low permeability soils using high rates of displacement, where peak strength may be unknowingly increased, or if feedback control is available, with the specimen height, and therefore volume, held constant. The fully drained tests necessary for interpretation of the results in terms of effective stress cannot necessarily be assured even at slow shear rates (see later, under test methods). After a period of neglect, however, the direct shear apparatus re-emerged as the apparatus of choice for determining residual shear strength (Skempton and Petley, 1967), and later as a method of determining the effective angle of friction of coarse granular materials used in reinforced earth. The issue of drainage and rate of shear need not be a problem in either of these applications.

The evident aim of traditional shear box testing is to induce failure along a pre-determined plane in the soil. However, the failure conditions in a shear box test are much more complex than might be thought, as Morgenstern and Tchalenko (1967) and Potts et al. (1987) have demonstrated. The ends of the box impose restraints that in intact clays initially produce high stress concentrations and pronounced edge structures, and these are then followed at failure by the development of an echelon sub-horizontal and sub-vertical discontinuities. Only at post-failure displacements do the edge structures and sub-horizontal discontinuities join up, forming an undulating but continuous shear surface. The overall effects of this are that:

- Progressive failure occurs, potentially leading to an underestimation of the strength of brittle materials.
- Because the failure surface is not smooth, planar and horizontal, energy is expended in dilation and in progressive remoulding of the high points of the surface. Very large displacements are required to remove the initial undulating geometry of the post-peak failure surface. Residual strength is better found using the 'cut-plane' technique described below.
- Interpretation of the results of shear box testing is uncertain.

As with most direct and simple shear tests, normal stress and shear stress are calculated for a single plane (the horizontal plane), actual stress conditions vary within the specimen due to the restraints imposed, and interpretation therefore depends upon the assumptions that are made. Routine shear box testing is interpreted on the basis that failure occurs on a horizontal plane, i.e. that the calculated shear stress and normal stress represent the conditions where the Mohr circle of stress touches the failure envelope of the soil. Morgenstern and Tchalenko's observations suggest that the central portion of the shear box specimen is in simple shear at failure, and that Hill's (1950) interpretation may be more appropriate when determining peak effective strength parameters.

When determining residual parameters, for the investigation of pre-existing slope instability, two situations typically arise. It may be possible to obtain block samples of the shear surface, from trial pits, in which case careful alignment of the shear surface with the centre of the box (and re-alignment if necessary after re-application of the vertical stress) will allow a good estimate of the available shear strength. Alternatively, and more commonly, the shear surface cannot be sampled, and the cut-plane procedure (Petley, 1966, Skempton and Petley, 1967) should be used since, although peak strength cannot be measured, this avoids the development of a wavy shear surface in the shear box (see below).

4.3.2 *The constant normal stiffness shear test*

The constant normal stiffness (CNS) shear test was developed (Lam and Johnston, 1982, Johnston et al., 1987) to mimic the increase in stress around piles socketed into weak rock. Essentially the apparatus incorporates a large version of a conventional direct shear box, so that actual or more commonly idealised joint geometries (Clayton and Saffari-Shooshtari, 1990, Gu et al, 2003) can be tested either bonded or unbonded on concrete. An initial normal stress is applied, and this is then locked off against a 'spring', so that the dilation of the interface under shear leads to increasing normal stress. In very weak rocks shear takes place through any asperities, but in stronger materials additional shear resistance is generated by the increased normal stress and the roughness of the interface. As might be expected, problems occur because of the need (for practical reasons) to restrict the length of the specimen. A large (1 m diameter) CNS ring shear apparatus has recently been constructed at the University of Sydney (Kelly et al., 2003) in an attempt to overcome some of the difficulties.

4.3.3 *Ring shear apparatus*

Originally used for shear testing during the very early days of soil mechanics (Cooling and Smith, 1936), the modern ring shear test was developed jointly between Imperial College, London and the Norwegian Geotechnical Institute (Bishop et al., 1971). This sophisticated apparatus can impose large relative displacements between the top and bottom of the apparatus, avoiding loss of soil by opening the gap only when measurements of shear stress are required. Perhaps because of its complexity and cost, the apparatus has not been widely used for commercial testing. It has, however, been used to great effect in developing our understanding of the residual strength of different soils (e.g. Lupini et al., 1981) and more recently of the effects of rate of shearing on residual strength (Skempton, 1985, Tika et al., 1996, Fearon et al., 2004).

Most routine investigations of residual strength carried out in recent times have used either the standard shear box (see above) or the simpler ring shear apparatus developed by Bromhead (1979). This consists of a 5 mm deep trough into which stiff soil is kneaded, and a porous annular plate (made of sintered bronze, and having a rough (machined) surface to encourage the development of a residual shear surface away from (below) the soil/platen contact. Drainage occurs at the top of the specimen. The apparatus has a gearbox to allow the rate of rotation to be varied, and as with other devices of this type, it is contained within a well, to prevent evaporation and control pore pressures on the top boundary. The test is described in BS1377:1990 Part 7, section 6. Care is needed in preparing specimens to ensure that coarse particle, which will disrupt the formation of a residual shear surface, are excluded. In this context 'coarse' should probably be interpreted as a particle size greater than about 0.1 mm (fine sand).

4.3.4 *The simple shear test*

The aim of the simple shear apparatus is to impose uniform simple shear deformation under plane strain, zero extension conditions. A number of types of apparatus exist, based on two early variants; the NGI apparatus (Bjerrum and Landva, 1966, Dyvik et al., 1987), and the Cambridge apparatus (Roscoe, 1953). The test is standardised in ASTM D6528.

The first direct simple shear apparatus was reported by Kjellman in 1951. Bjerrum and Landva modified this apparatus so that undisturbed specimens could be tested, using a wire reinforced membrane to maintain zero lateral strain conditions during shearing. They reported that the simple shear test gave results that were closer to those deduced from back analysis of some field failures, and from the field vane test, than were the results of triaxial testing. As with the direct shear test described above the apparatus has the merit of being relatively simple, the specimen required is small (typically only 70 mm diameter) so that large undisturbed samples are not required) and thin (20 mm high) so that consolidation occurs relatively rapidly. In addition, the mode of shearing involves principal stress rotations similar to that imposed in many practical geotechnical situations (Wroth, 1987), and which are associated with reductions in strength (Symes et al., 1984).

Bjerrum and Landva were not able to perform truly undrained tests in the direct simple shear apparatus. As a result, although most tests are performed to obtain undrained shear strength, the undrained simple shear response of soil is normally investigated by performing drained tests at constant specimen height, modifying the vertical load during shear. The changes in vertical stress that are measured are assumed to correspond to the pore pressures that would be measured in an undrained test. This has been verified for firm clay specimens (Dyvik et al., 1987).

Despite its evident attractions, the simple shear test has been criticised on a number of counts:

- Unless special measure are taken (e.g. pins, which may cause sample disturbance), slippage may occur between the specimen ends and the platens.

- Because of the way load is applied, the stresses in the soil element under test are not uniform.
- The method of restraint (i.e. by a reinforced rubber membrane, a ring stack, or a rigid platen) cannot induce uniform straining without strain localisation occurring.
- Because of the above, and the limited measurements that are made, it is difficult to interpret the results in a manner comparable with that of other tests (e.g. by constructing a Mohr circle of stress at failure).

The application of shear force to the top and bottom (but not the sides) of the specimen requires non-uniform normal stresses at the boundary, to provide a resisting moment. Experiments with real soil specimens (Vucetic and Lacasse, 1982) with different aspect ratios, and measurements of stress on the boundaries, suggest that the results of tests on clay specimens can be interpreted with more confidence than those on sands.

4.4 *Triaxial loading*

Early development of triaxial-type tests can be traced back Terzaghi (1932) and to the Dutch ‘cell test’ which was in use in the Delft laboratory from the 1930s (Geuze and Tan Tjong Kie, 1950, van Duren and Heijnen, 1979). Important early contributions were also made by a number of others, for example Taylor (1939, 1948). However, the publication in 1957 of the first edition of Bishop and Henkel’s classic work on ‘The Measurement of Soil Properties in the Triaxial Test’ provided easy access for practitioners to the apparatus, instrumentation and techniques needed to produce high-quality soils data on a range of materials, and had a huge impact on laboratory testing around the world, providing the basis for what must now be regarded as ‘routine’ triaxial testing.

4.4.1 *Routine triaxial testing*

In countries where there is a tradition of soil testing, the triaxial test is the standard method of determining not only the undrained shear strength of soil, c_u (also s_u and τ_u), but also its effective strength parameters, c' and ϕ' . The apparatus takes much the same form all over the world, but details vary. In general (Figure 5) the cell has a metal base, to which is attached a specimen pedestal. Ducting enters the triaxial base, and can be connected (usually through two separate ports) to the base of the specimen. Further ducting and tubing may allow access to the top of the specimen via the top cap. An access ring can be fitted between the base and the cell top, to allow for additional instrumentation. The specimen, which typically can have a diameter of 38 mm, 50 mm, 70 mm or 100 mm, depending upon the country where the test is to be carried out, normally has an aspect ratio of about 2. It is sealed by o-rings in a rubber membrane, and surrounded by fluid (normally water) contained in the cell top.

The common usage of the triaxial apparatus is associated with its important advantages. It is relatively easy to prepare a cylindrical specimen, especially if this has the same diameter as the sample from which it is taken. Because the specimen is sealed by base pedestal, rubber membrane, and top cap, global drainage can be controlled, and displacements, pore pressures or volume changes measured. Although uncertain stress patterns result from platen friction, the length to diameter ratio of the specimen (generally two, to minimise platen friction effects in the mid-section of the specimen) allows sufficient space for local instrumentation to be attached.

However, the test has a number of disadvantages. The amount of material required for a test specimen is large relative, for example, to that required for the oedometer, direct or simple shear tests. Platen friction causes the development of uneven total stress distributions at the end of the specimen, leading to barreling and strain localisation at failure and excess pore pressure variations both longitudinally and radially in undrained tests. Loading can only take place under axi-symmetric conditions in what is sometimes referred to as the triaxial plane, where the principal stresses are vertical and horizontal. With the exception of 90° stress jumps (when moving, for example, from triaxial compression to extension) principal stress rotation is not possible.

Triaxial cell construction varies, but in Europe generally (Cavallaro et al., 2001, Georgiannou, 2001) consists of a Perspex cylinder fitted to a metal cell top. All round pressure (cell pressure) is applied through a cell fluid, which is generally water. Even though it might sometimes be thought advantageous (for example to protect electronic instrumentation) to use air or some other gas at the top of the cell, health and safety considerations should prevent this. The amount of energy stored in a gas-filled cell is very large; failure of the Perspex cell wall can lead to an explosive rupture and violent discharge of cell parts around the laboratory.

Load was originally measured using a proving ring, mounted outside of the cell between the ram and the cross arm of the compression machine. This meant that the measured deviatoric load included not only the load applied to the specimen and also the uplift force due to the action of the cell pressure on

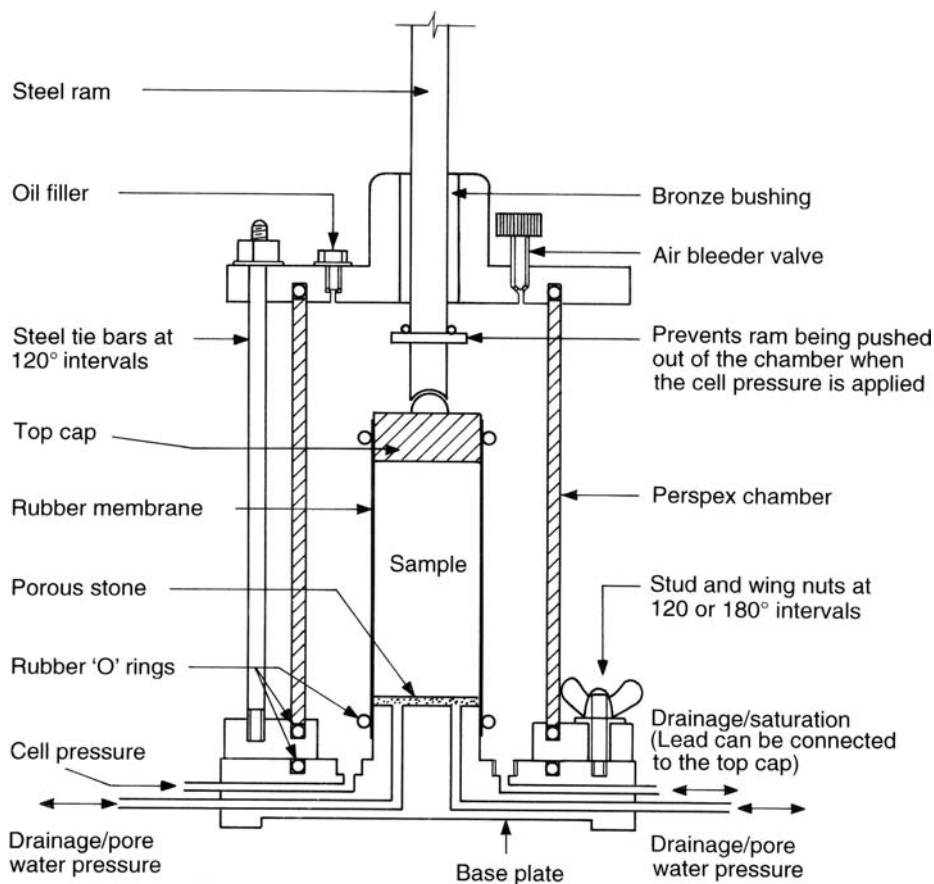


Figure 5. Typical triaxial cell in use in UK commercial testing.

the base of the ram (which in a test with constant cell pressure could be zeroed out), but also any ram friction. Some early triaxial cells had loose fitting rams in an attempt to control ram friction. The top of the cell would be filled with viscous castor oil, to reduce leakage during testing. This was unpleasant to work with, and led to crazing of the internal surface of the cell.

In most conventional triaxial compression testing, deviatoric loading is applied through a ram that enters the top of the cell through a bushing, and pushes against the top cap of the specimen, described here as a “free” connection. A number of other top platen connection arrangements are now in use, in response to the need to apply upward deviatoric force in extension tests, or because of other cell designs:

- The ‘hard lock’ system, where the top platen is screw-threaded to the ram, with or without a locking nut.
- A similar system, but with a captured ball, allowing some rotation between the ram and the top platen.
- The ‘suction top cap’, where a silicon rubber or other attachment is used in conjunction with a vent to atmospheric pressure to cause the bottom of the ram to contact positively with the top platen.
- Magnetic systems, where a coil on the end of the ram creates attraction to a steel top to the platen (Figure 6).

In the UK, as described above, the ram butts on to the top cap. The arrangements most commonly adopted are those of a hemispherical ram end bearing onto a dimpled top cap, or of spherical ball between a dimpled top cap and ram with a conical seating (Figure 6). In contrast (and in common with the test arrangement in some other forms of apparatus, for example the Bishop and Wesley stress path cell when used with a suction top cap, for the triaxial extension test) Japanese Geotechnical Society standards for consolidated undrained and consolidated drained triaxial tests require a rigid connection between the top cap and the loading ram, in what is sometimes referred to as an ‘overcloak’ cell (Figure 7). Baldi et al.

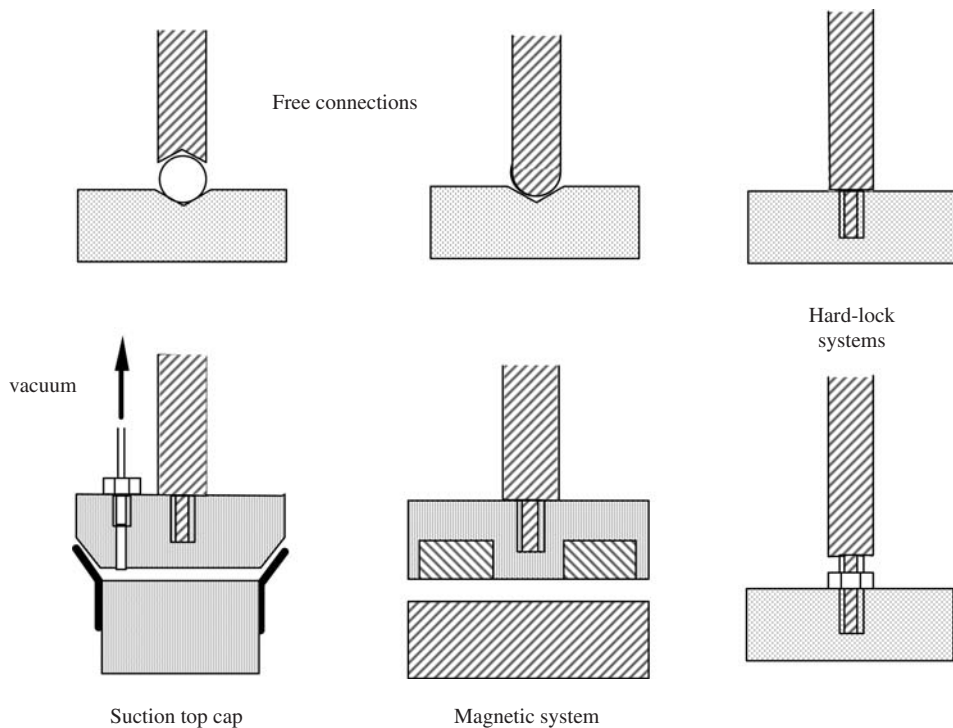


Figure 6. Top platen connections.

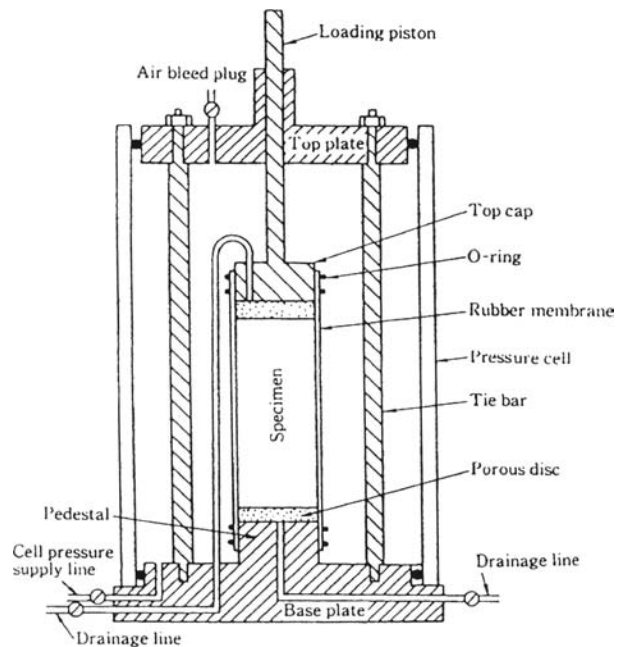


Figure 7. Japanese 'overcloak' cell, with rigid piston / top platen connection (Kuwano et al., 2001).

(1988) note that this apparently minor detail can have a significant effect on the stiffness measured at small strains.

Even with a loose fitting ram (BS1377:1990 specifies quite sensibly that 'friction between the piston or seal and its bushing shall be small enough to allow the piston to slide freely under its own weight when the cell is empty'), it was found that friction losses could be high (Bishop and Henkel quote a maximum of about 5% of the measured load) if the top of the specimen attempted to move laterally. This can occur towards the end of a test where failure has occurred on an inclined shear surface. In the UK the response to this problem was to use a rotating bush to reduce the frictional force on the ram. Probably because of the complexity and size of the system, and concerns about rotating bushes applying torque to the top of the specimen, this practice eventually became discontinued around the time that electronic load cells were developed at Imperial College and became available to test houses and other researchers.

At the time of the introduction of the internal load cell it seems to have become the practice to fit an 'o' ring inside the piston bush in order to prevent water leakage from the cell. If such a system is used in conjunction with an internal load cell this does not create a problem, but in many labs one can now find extremely stiff cell rams (which do not meet the BS criterion) being used in conjunction with external load measurement.

The entire cell is mounted within a compression machine that is designed to push the ram into the top of the cell at a constant rate, here termed the 'machine rate of deformation'. The accuracy with which the machine rate of strain can be set has been found to vary quite considerably from machine to machine. This can be a significant issue, for example when attempting to control the set up of excess pore pressure in a constant rate of strain consolidation test.

A great many variations in the detail of the apparatus exist.

- i. For unconsolidated undrained triaxial testing the specimen base pedestal should, ideally, not have ports or porous stones (see below). There are no requirements either for drainage or pore pressure measurement, and the inclusion of porous stones increases specimen compliance whilst allowing water content changes to occur.
- ii. In routine effective stress testing pore pressure is generally measured at the base of the specimen using a pressure transducer mounted in a de-airing block attached immediately to the side of the triaxial base.
- iii. Porous stones are used between the base and any top cap port and the specimen, to allow drainage from the majority, or all, of the surface whilst preventing blockage of the ports by soil. In routine testing porous stones are usually coarse carborundum (silicon carbide) discs that are boiled in water between tests, to clean and de-air them.
- iv. Drainage against a 'back pressure' can take place either through the second port in the base, or through a top cap port and porous stone. In routine testing, it is better if possible to use a blank top cap, without drainage ports, since it can be difficult to ensure complete saturation of the port and a coarse porous stone when mounting these on top of the specimen. However, lack of top drainage increases specimen consolidation periods, and therefore may extend the overall test period unacceptably.
- v. Specimens may be tested with side filter drains if it is necessary to speed the progress of the test, and in particular any consolidation or swelling stages.
- vi. In special tests, and particularly tests where the soil has an initially high suction, high air entry porous discs should be used in order to reduce the amount of water taken in by the specimen during set-up and instrumentation, and to avoid desaturation of the porous stones and ports. High air entry ceramic discs (with an air entry value of about 200 kPa or higher) should be flush mounted in the base pedestal (and in the top cap if drainage from the top of the specimen is required), and two ports should be provided to the cavity behind each stone in order to allow it to be flushed. The space between the periphery of the stone and the pedestal or top cap should be filled with epoxy resin.

However precisely prepared and mounted a specimen of stiff clay may be, experience shows that the initial application of cell pressure often causes fissures to close, and the alignment between the ram and top cap to be lost. A conventional dimpled top cap forces lateral movements of the top of the specimen during the early stages of loading, leading to specimen bending, a reduced stiffness (even when local strains are measured) and (in extremes) stick-slip behaviour. This can be improved by using a hemispherical ram end bearing on a flat top cap. However Baldi et al. (1988) showed that below about 0.3% axial strain the locally measured stiffness was higher when a rigid connection was used, even though a flat-ended ram bearing on a halved steel ball was used to avoid alignment problems.

ASTMD3999 therefore places limits on the lack of alignment between platens specimen and the loading ram (Figure 8).

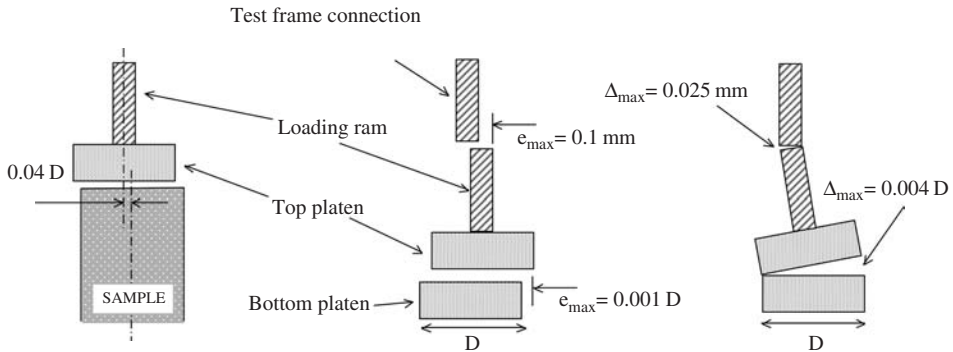


Figure 8. Limits on acceptable platen and load rod alignment (ASTM D3999-91).

4.4.2 Cyclic triaxial testing

Cyclic triaxial testing has a long history, but primarily in those regions that are seismically active or to provide data for dynamic loading situations (JGS Standard 0541-2000, ASTM D3999). Thus cyclic triaxial compression tests are unusual in the UK, but apparently quite common in Japan (Kuwano and Katagiri, 2001), and standardised in the USA (ASTM D3999), where this is an important method of stiffness measurement at intermediate axial strain levels (greater than about 0.01%). The apparatus most used is basically the same as a conventional undrained triaxial compression cell, but with a mechanical arrangement that allows the deviatoric load to be cycled. Standards and facilities also exist for carrying out cyclic torsional shear tests, although it appears that in most countries these are much less commonly used.

In most applications it is the load/unload modulus that is obtained from the test, using in-cell (but not local strain) deformation measurement (Kuwano et al. (2001), Georgiannou (2001)). Based on the first authors' experience of testing rail formation material in the cyclic triaxial compression apparatus, and observing the stiffness determined by weak rock pressuremeters, it seems likely that the stiffnesses obtained by these systems will (because of bedding) be low when the material under test is a hard soil or soft rock. Torsional tests appear preferable.

4.4.3 Stress path testing

Lambe (1967) developed the concept of the stress path test in response to the need to determine stiffness parameters for materials whose response is clearly most often stress-path dependent. Although the stress path that will be experienced by an element of soil is a function of both the stiffness profile of the ground as well as loading/unloading levels and geometry, Lambe noted that for shallow problems stiffness is a secondary issue. The stress path that will be undergone by a soil element can be predicted with reasonable accuracy, and applied to a representative soil specimen to determine its stress-strain response under the given loading path. By selecting key representative locations, laboratory stress path testing can be used to obtain the representative stiffness of soils for numerical modelling, for example using finite element or finite difference analyses.

The Bishop and Wesley stress path cell (Bishop and Wesley, 1975), designed to apply stress control under triaxial loading conditions, is quite commonly in use around the world. The apparatus uses rolling diaphragm seals and a linear bearing to deliver axial force to the base pedestal.

The apparatus, although complex and therefore relatively expensive, allows easy stress path control, although it has some characteristics that need to be borne in mind:

- The top of the specimen bears against the cell top, thus further loading the tension bars that attach it to the base of the apparatus. External strain displacement measurements made between the cross arm and the cell top will include the effects of this, in addition to errors caused by load cell compliance, filter paper and (when used) lubricated end platen compression, bedding between porous stones, platens and the specimen, and the effects of plated friction. Local strain measurement would seem essential, since the apparatus is generally used for advanced or high quality testing.
- The upper rolling diaphragm, which acts to contain the cell fluid, tends to collect debris from specimen preparation. Static friction can become quite large, and the equipment should therefore be checked regularly for this. Even when well maintained, rolling resistance will be significant in some applications, and must be taken into account either by using an internal load cell for feedback, or by correcting stress paths after the test. Bishop and Wesley quote a value of 4.2 N as the observed

difference in bottom chamber pressure when moving up and moving down, equivalent to a principal stress difference of 3.7 kPa.

The stress path cell can also, in principal, be used to carry out strain controlled testing. However, because of the effects mentioned above, and in addition because the rolling diaphragm convolute strains as it rolls following a change in chamber pressure, strains determined on the basis of volumes (for example pumped into or out of the lower chamber) are unlikely to be reliable. Local strain measurement is required.

4.4.4 *Resonant column apparatus*

Usage of the resonant column apparatus varies very considerably across the world. In the UK, probably because of a lack of seismic activity in the region, it is unusual, with perhaps fewer than 10 existing in the country. In Japan about 20% of the 92 organisations responding to a nationwide questionnaire in 1995 owned a resonant column apparatus (Kuwano and Katagiri, 2001). The equipment, used to determine the very small strain stiffness and damping of soils, has a number of variants. All aim to produce single degree of freedom vibrations in the specimen at controlled frequencies and variable (although always small) amplitudes. These are typically measured using an accelerometer mounted on the top platen.

In the Hardin oscillator a spool-shaped spring couples a central mass and the specimen top cap to an external hollow cylindrical mass with large rotational inertia. The oscillator has four magnets attached to it, which are driven by electrical coils to obtain the desired torsional vibrations. The large inertial mass of the oscillator essentially provides a reaction such that the forcing torque delivers vibration to the top of the specimen. It can be used within a fairly conventional triaxial apparatus, allowing specimens to be both isotropically and anisotropically consolidated before they are tested.

The Stokoe oscillator is simpler. The coils are mounted on a frame attached to the base of the apparatus, which provides drive reaction. This arrangement can provide greater torsional moment, and therefore higher torsional strain, but specimens cannot be loaded anisotropically except through the insertion of a wire from the top cap, through the central axis of the specimen, then through a basal gland to an air actuator or hanger loading system.

The Drnevich Long-Tor resonant column combines a torsional system with a longitudinal electromagnetic actuator, allowing the determination of compressional stiffness. A relatively simple modification of the Stokoe device (Cascante et al., 1998) allows specimens to be tested in flexure, from which the flexural value of Young's modulus can be determined. Comparative testing on fine sand and clayey silt using a number of different types of resonant column apparatus is reported by Skoglund et al. (1976).

In all tests the amplitude of the torsional vibration of the specimen is monitored as a function of frequency. Shear modulus is determined from the specimen properties, the resonant frequency and apparatus constants. Damping can be determined by observing free vibration decay (good at higher amplitudes) or using the half power method (which provides better results at lower amplitudes when the signal to noise ratio may be a problem).

A number of problems and uncertainties have been noted with the resonant column apparatus.

- The rate of free vibration decay is increased by the effects of back EMF in the drive coils, and thus reported damping will be higher than would be produced by the test specimen itself. Back EMF effects can be removed by making the drive system open circuit, but measures must be taken to cope with the large transient voltages that are then produced.
- The test is interpreted on the basis of a single degree of freedom. In reality the various parts of the system are not rigidly connected (Min et al., 1990) or constrained (Ashmawy and Drnevich, 1994), and the system is somewhat compliant.
- A single accelerometer is used to record the angular displacement of the top of the specimen. For specimens with an aspect ratio of the order of two torsion should be the first mode of distortion, but for more slender specimens this might not be the case.
- End effects may become important as specimens become stiffer. Slippage between the specimen and the platens may occur if excessive torques are applied.

A number of resonant column variants are now available that also offer a torsional shear option (see for example Nishimura et al., 2006). Slow shearing requires different displacement instrumentation, typically provided by proximity transducers.

4.5 *Advanced loading systems*

We are limited in our ability to measure the anisotropy of strength and stiffness in soils, by controlling both the direction of the major principal stress, σ_1 , which can be defined in terms of its angle to the

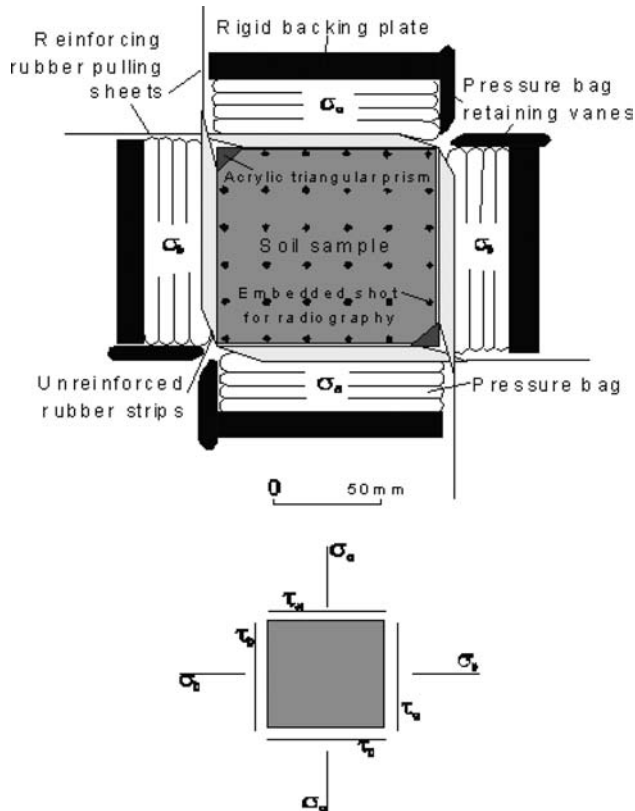


Figure 9. Application of normal and shear stresses in the UCL Directional Shear Cell (Arthur et al., 1981).

vertical, α , and the relative magnitude of the intermediate principal stress, σ_2 , which is defined in terms of $b = (\sigma_2 - \sigma_3)/(\sigma_1 - \sigma_3)$. In the triaxial compression test α equals 0° and b equals 0, whereas in the triaxial extension test α equals 90° and b equals 1. Comparison of data from the two types of test on the same soil does not allow the effects of changes in α and b to be separated. A number of pieces of advanced soil testing apparatus have been developed to allow this.

The principles involved can be understood by considering three specific pieces of apparatus; the Directional Shear Cell (DSC); the plane strain compression apparatus with flexible boundaries (PSC), used to test tilted samples, i.e. samples in which the initial direction of the bedding relative to the fixed direction of σ_1 can be varied; and the Hollow Cylinder Apparatus (HCA).

The DSC, introduced by Arthur et al. (1977 and 1980), is a plane strain device in which the direction of the major principal stress is controlled during shear by applying both normal stress and shear stress to four faces of a cube (Figure 9). The DSC can be used in two configurations, as shown in Figure 10. In the first (Figure 10(a)), it can be used to shear samples in a plane parallel to the bedding planes. In the second configuration for the DSC, shear and normal stresses can be applied to control the principal stress directions in the vertical plane, i.e. the plane containing the bedding planes (see Figure 10(b)) to examine the initial or inherent anisotropy.

In the hollow cylinder apparatus, a sample for which is shown in Figure 11, control over the major principal stress direction, α , and of b , is achieved by independently varying the inner and outer cell pressures, p_i and p_o , the axial load, W , and the torque, M_T . This is the only apparatus in which both α and b can be independently controlled.

Stress and strain non-uniformities are inevitable in a hollow cylinder specimen under test as soon as a torque is applied, since this generates a shear stress across the specimen wall that varies with radius. This non-uniformity increases when different internal and external pressures are applied, in order to separate changes in b and α , and because of the effects of end restraint. The geometry of a hollow cylinder specimen has to be selected to minimise these non-uniformities. The first requirement is to keep the ratio of the inner and outer radii as close to unity as possible, effectively to minimise the curvature. In

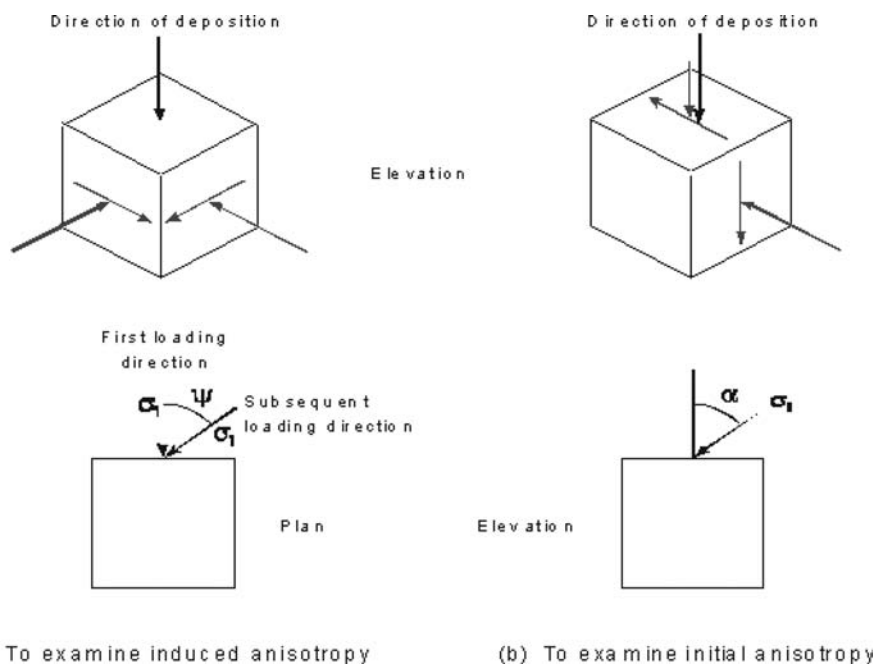


Figure 10. Configurations and uses of the Directional Shear Cell.

the first HCA developed at Imperial College (Hight et al., 1983) inner and outer radii of 100 mm and 125 mm were used, i.e. a ratio of 0.8.

The minimum wall thickness that can be used will depend on a number of factors, to ensure that:

- Failure mechanisms are not constrained.
- A sample volume is large in relation to any potential volume change resulting from membrane penetration.
- Granular specimens can be pluviated with a uniform density across the wall.
- Clay samples are large in relation to the potential thickness of zones of disturbance created during preparation.
- Local instrumentation for displacements and pore pressures does not significantly affect measurements.

Hight et al. (1983) tested Ham River Sand, which had a D_{50} of 0.4 mm and a maximum particle size of 0.7 mm, in a cell with a wall thickness of 25.4 mm (giving a ratio of 36 between the wall thickness and maximum particle size, although they envisaged that sand with a maximum grain size of 1 mm could be accommodated). The specimen height was set at 250 mm, i.e. the same as the outside diameter of the specimen on the basis that all measurements of displacement and pore pressure would be made internally over a central gauge length. With these dimensions the limits on the combinations of α and b that can be investigated without unacceptable levels of non-uniformity are very approximately $b > 0.7$ with $\alpha < 15^\circ$ and $\alpha > 75^\circ$ with $b < 0.2$, (but see Hight, 1983 for a full discussion).

Saada (1988) conducted a comprehensive review of the advantages and limitations of the HCA. Criteria for the dimensions of HCA specimens were proposed as:

$$H \geq 5.44 \sqrt{r_o^2 - r_i^2}$$

and

$$n = \frac{r_i}{r_o} \geq 0.65$$

It is difficult to overcome end restraint in an HCA because of the need to apply a torque to the specimen. Recent analyses reported by Rolo (2003) indicate that even when monitoring over a central

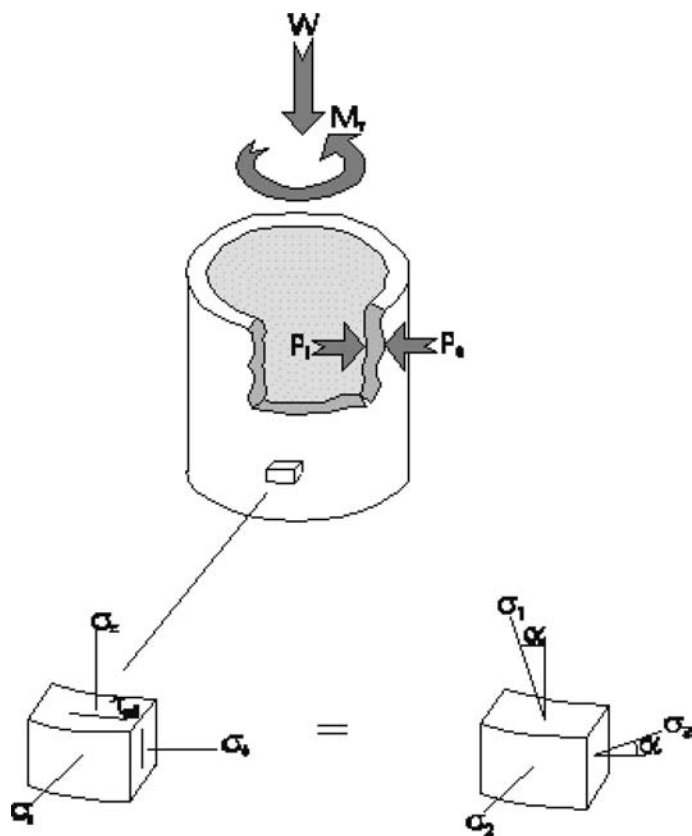


Figure 11. Idealised stress conditions in the wall of a hollow cylinder apparatus.

gauge length, a height to outside diameter ratio approaching two is preferable. With an H/OD of two Rolo (2003) has shown that the effects of non-uniformities are reasonably small when operating with a ratio of inner to outer radii of 0.6. The Mark II ICHCA has an inner diameter of 60 mm, an outer diameter of 100 mm and a height of 200 mm, the same dimensions as the cyclic HCA at Southampton University. These smaller dimensions make procurement of intact samples less problematical.

Not being an element test, results of an HCA test must be interpreted as a boundary element problem and assumptions made as to the average stresses and strains across the wall of the specimen. A set of assumptions used in interpreting tests in the IC HCAs is given by Hight et al (1983). In view of the need to make these assumptions and the additional non-uniformities introduced by end restraint the HCA should be regarded as a tool for phenomenological and comparative studies, rather than parameter determination.

The ability of the HCA to simulate rotation of principal stress directions has led to the increasing use of the apparatus in the study of the behaviour of ballast and formation materials for rail track and road pavements (Brown, 1996, Grabe, 2002).

4.6 Testing equipment for unsaturated soil

Early interest in the behaviour of unsaturated soil in the UK (e.g. Croney et al., 1952, Black et al., 1958) was driven by a desire at the Road Research Laboratory to provide a scientific background to the design of road pavements. New techniques developed in the late 1950s and 1960s appear to have been associated with the design of large earthfill dams. There has been a subsequent resurgence of interest as it has become appreciated that unsaturated soil mechanics is relevant in a very wide range of applications, including landfill covers, residual soils, low rise building foundations, and shallow linear infrastructure such as pipelines and highway and rail formations (for example see Ridley et al., 2003).

The equipment used to test unsaturated soils is broadly similar to that used for saturated soil, but with the important additions of methods to control or measure matrix suction. Of the two possible ways to

control net normal stress ($\sigma - u_a$) and matrix suction ($u_a - u_w$), the axis translation method (Hilf, 1956, Bishop and Blight, 1963) has proved the easiest to implement to date. A key element of this success was the introduction of flush-mounted flushable high air entry (HAE) porous stones in the base of a range of apparatus. As noted by Bishop and Henkel (1962 – Appendix 6) pore water pressures can only be measured if a number of conditions are met:

1. To prevent cavitation of the water in the measuring system, the pore water pressure must be kept above about 80 kPa or 90 kPa. If cavitation occurs, the specimen will be able to draw water out of the porous stone.
2. To prevent air from entering the porous stone at the start of a test the initial value of negative pore water pressure in the specimen (before any application of all-round total stress) must not exceed the air entry value of the stone. If this is not so, then the stone will suck in air and desaturate as the specimen imbibes water from it.
3. After the specimen has been set up, and subjected to increases in pressure, the matrix suction should at no stage exceed the air entry value of the porous disc.
4. Air which diffuses through the porous stone with time is kept to a minimum by flushing the cavity beneath the stone.

In early apparatus, such as that described by Bishop and Donald (1961), high air entry stones for measurement or control of pore water pressure occupied much of the apparatus pedestal, and pore air pressure was measured or controlled from the top or sides of the specimen. More recently, double-drainage systems, with a central HAE stone and a peripheral coarse annular coarse stone have been used on low-permeability soils (Romero, 1999, and Barrera, 2002, quoted in Hoyos et al., 2005).

In the triaxial apparatus an inner cylinder has been used both to prevent air migration and to allow measurement of volume change (Bishop and Donald, 1961, Ng et al., 2002). Double-walled cells have also been used to avoid errors in deducing specimen volume change from cell volume change (e.g. Wheeler, 1988).

The axis translation method has been used in oedometer testing (e.g. Escario and Saez, 1973 who used a cellulose membrane that was impermeable to air but not to water), in direct shear testing (Escario and Saez, 1986, De Campos and Carrillo, 1996) in torsional shear testing (Mancuso et al, 2000), and most recently in triaxial testing by Cabarkapa and Cuccovillo (2006). A number of other special features have been included in apparatus for testing unsaturated soils. For tests where the specimen is not sealed in a membrane, for example the shear box test, the axis translation technique has been implemented by enclosing the apparatus in a air-pressurised chamber, and fitting a HAE stone in the base (e.g. Gan and Fredlund, 1988, Feuerharmel et al., 2006).

The second method to control matrix suction is by osmosis and osmotic control. This has been used in oedometer testing by Cui and Delage (1996) and Dineen and Burland (1996), and in triaxial testing by Cui and Delage (1996).

Direct suction measurement using tensiometers is attractive, and is the subject of recent and current research (Ridley and Burland, 1993, Take and Bolton, 2003). Ridley and Wray (1996) provide a review of methods suction measurement in the laboratory and in situ. Instrumentation is discussed under Measuring Systems and Ancillary Equipment, below.

Since the pore air pressure is atmospheric in most practical problems (Fredlund and Rahardjo, 1993) the soil-water characteristic curve (SWCC) has increasingly been used as a basis for predicting other geotechnical properties, such as hydraulic conductivity, shear strength and stiffness. A range of apparatus and techniques is available for the determination of the SWCC, using for example the pressure-plate apparatus (Wang and Benson, 2004), the suction-plate apparatus (Feuerharmel et al (2006), and the filter paper suction method (Chandler et al., 1992). A recent example of the complexity and care required for the accurate determination of suction for two Brazilian colluvial soils with bimodal SWCCs, including an excellent review of the factors influencing the results of the filter paper method, can be found in Feuerharmel et al (2006). A striking feature of these methods, highlighted by Ridley and Burland (1993), is the significant time required to make most suction measurements.

5 MEASUREMENT SYSTEMS AND ANCILLARY EQUIPMENT

The overall accuracy of a measurement is difficult to determine, since it requires knowledge of the true value of the quantity to be measured, the measurand. According to Dunnicliff (1993) accuracy is the ‘closeness of approach of a measurement to the true value of the quantity measured’, but this definition

is then followed by the statement that it can be ‘evaluated during calibration, when the true value is the value indicated by an instrument whose accuracy is verified and traceable to an accepted standard’, which seems to imply that accuracy, in the sense used by Dunnicliff, is unaffected by conformance – the effect an instrument may have (simply due to its presence) on the quantity to be measured.

‘Overall accuracy’ is here used to mean the closeness of a measurement to the true value of the quantity to be measured at a required measurement position, had the instrument not been present. It depends upon:

1. *Resolution*, which is the smallest increment that the recording system, typically using analogue to digital conversion at some stage during data capture, can deliver;
2. *Precision*, which is a measure of the variation in the data captured by a monitoring system when the instrument is stationary, and the true value is not changing. Such variations are typically caused by random effects such as electrical noise.
3. *Repeatability*, which includes the above effects and also any systematic but unquantified effects, such as temperature effects.
4. *Intrinsic instrument performance*, for example non-linearity, hysteresis and temperature sensitivity (where known) of a transducer sensor.
5. *Conformance*. The presence and position of an instrument can modify the value of the parameter being measured. This generally occurs in one of two ways. Firstly, the placement of an instrument can modify the quantity to be measured (for example, a ‘soft’ pore pressure measuring system may reduce the change in pore pressure that occurs in a specimen). Secondly, the coupling between the instrument and the soil skeleton or its pore water may be inadequate. Examples for soil testing include the use of external displacement transducers to measure strain, and the use of base mounted pore pressure transducers to measure mid-plane pore pressure.
6. *Consistency of datum values*. Some systems are required to measure relative values, and require a common datum. For example, the accurate and consistent determination of effective stress requires all pressure transducers (e.g. for cell pressure and for pore pressures possibly located at a number of positions on the specimen) to be calibrated to the same datum level.

Since most instruments have outputs that are to some extent temperature dependent, and many advanced laboratory tests on soil can take days or weeks to perform, a temperature controlled laboratory environment would seem essential. It will also need to be equipped with suitable calibration equipment for the sensors that are in use. Regular calibration (which is much less frequently carried out for commercial testing than would seem desirable), showing a consistency of output, builds confidence in instrument performance but also helps identify when (as is inevitable) instruments fail. Instruments used for research tests should ideally be fully calibrated (or receive a calibration check) before and after each test. Calibration records need to be systematically examined, and filed to allow comparison with future calibration results.

The quantities to be determined during soil testing are, fundamentally, time, mass, stress, pressure and strain. These are often determined indirectly, so that in reality the following measurements are needed:

1. Time;
2. Mass;
3. Dimensions – area, length, etc.;
4. Load;
5. Fluid pressure;
6. Displacements – local and global;
7. Volume change.

5.1 *Assessing required instrument accuracy*

In the relatively simple case of pore pressure measurement, it is desirable (given the magnitudes of the effective stresses in many geotechnical problems) to achieve an accuracy of better than 1 kPa. The Perspex triaxial cells commonly used for routine testing are generally rated for cell pressures of about 1500 kPa. Testing may require back pressures of up to 800 kPa, although lower values (say 300 kPa to 500 kPa) are more common, allowing isotropic stresses in the range 0–1000 kPa. It is obviously desirable to standardise on the type of pore pressure transducer that is used, but there is some difficulty if a top pressure of 1500 kPa is to be measured, since the required accuracy will then be $1/1500 = 0.067\%$ of the full range. In the case of pore pressure it is the absolute measurement (rather than the difference between the current value and the starting point) that needs to be within the required value. For example, the

combined non-linearity and hysteresis of a high-quality pressure transducer such as the Druck PDCR810 is only quoted as 'better than 0.067%'. The overall accuracy of pore pressure measurement will be lower, given the additional inaccuracies added by (for example) instrument amplification, drift, and particularly conformance.

Consider the determination of small strain stiffness (for example, see Clayton and Heymann, 2001). To achieve a measurement of 0.01% axial strain on the central third section of a 100 mm diameter $\times$ 200 mm high specimen requires the measurement of a relative displacement (over the gauge length) of 7 μm for a single gauge, or about 14 μm if two gauges are being averaged. Even if the stiffness is required to have an accuracy of only 10% of the measured value (which would normally be considered an extremely low accuracy) then the local strain gauges must have an accuracy of at least 1.4 μm , and a resolution somewhat better than this. Now consider the requirements of load measurement in an undrained test carried out to determine small strain stiffness. When testing London clay (for example with a maximum undrained Young's modulus at very small strain of 250 MPa) the deviatoric load at 0.01% strain might be of the order of 160N. To achieve a comparable level of accuracy as that required from the small strain measurement the internal load cell should have an accuracy of better than 16N, and a resolution somewhat finer than this. The combined inaccuracy in stiffness resulting from two measurement systems each with a 10% error could then be as large as about $\pm 20\%$.

Achieving an accuracy of deviatoric load measurement better than 16N should be no problem, given that a good submersible internal load cell used in this application might have a range of 5000N and a combined non-linearity and hysteresis better than 10N, even when loaded and unloaded over its entire range (as opposed the better performance that could be expected when it is loaded over a much smaller part of its range). Clearly the local strain gauges are the critical instruments in this stiff clay test, and all the more so because their coupling to the specimen leaves uncertainties about the gauge length to be used in computing local strain.

Next consider a test on a soft clay specimen. For this example the undrained Young's modulus at 0.01% strain is taken as 15 MPa, which is similar to that of a high quality specimen of Bothkennar clay. The requirements for local strain measurement remain the same, but the need for accuracy in load measurement increases. To achieve a comparable level of accuracy as required from the small strain measurement the internal load cell should now have an accuracy of better than 1.2N. Thus stiff materials such as heavily overconsolidated clays and weak rocks require greater accuracy of local strain measurement, whilst soft materials require greater accuracy of deviatoric load measurement.

In the case of the soft clay example, resolution also becomes an important issue. Although modern data acquisition systems typically use 16-bit or 32-bit analogue to digital converters, giving a dynamic range of at least 65,536, the useful resolution is greatly reduced by noise and by the use of fixed amplification ranges. It is unusual to achieve a resolution very much better than 1/4000 of the full range of an instrument, which in this case would be equivalent to 1.25N. Since the resolution of a measuring system must always be greater than or equal to accuracy, it would be necessary to replace the load cell with one more sensitive, or to calibrate it over a reduced load range, if greater accuracy or the measurement of stiffness at smaller strains were required.

In each and every test, instrument requirements should be questioned. In the example given above the acceptance of $\pm 20\%$ inaccuracy would seem far too lax, given that additional factors will increase errors further. But from the figures given above it can be seen that achieving better instrument accuracy will be a challenge. This simple example shows that the accuracy required of instruments varies not only with the type of test, but also with the material tested and stress and strain levels at which results are required. Measurements must have reasonable accuracy at axial strains of about 0.001% (Tatsuoka and Shibuya, 1992, Clayton and Heymann, 2001) if E_{umax} is to be determined during triaxial testing. This is only just achievable with the greatest care using current triaxial measuring systems.

5.2 Instrumentation and its calibration

For advanced testing the most common types of sensor are used for displacement, volume change, pressure and load measurement. In a few types of apparatus other instrumentation has been used, e.g. direct measurements of normal and shear stress in the Cambridge simple shear apparatus (Airey and Wood, 1987), but this is unusual.

5.2.1 External displacement measurement

Displacement measurement is used in the direct shear apparatus, and as the basis of local and external strain determination in the triaxial apparatus. Because of the effects of bedding and compliance, external displacement measurement does not need to be made to very great precision.

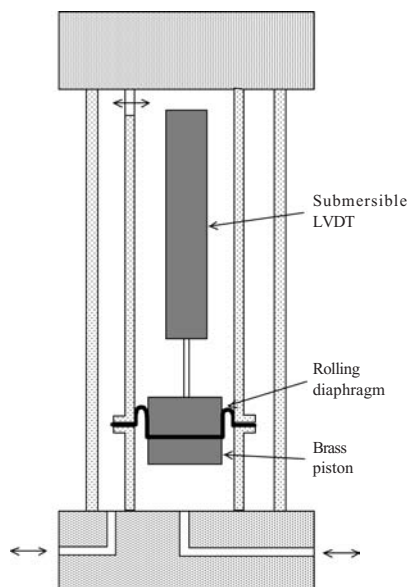


Figure 12. Rolling diaphragm electronic volume change gauge (Menzies, 1975).

Until the 1980's displacements were generally measured using dial gauges, and this remains the case in many laboratories. It is generally assumed that dial gauges do not require calibration, and the authors have rarely seen systematic attempts to assess this type of instrument in the same way that electronic transducers are normally calibrated. The traditional method of traceably calibrating displacement dial gauges uses steel gauge blocks ('slip gauges') (BS EN ISO 3650). Digital dial indicators have sometimes been used as a replacement for traditional mechanical displacement dial gauges.

Electronic external displacement measurement typically uses either LVDTs or similar devices. For reasons of stability, in the UK the preference for high quality long-duration testing still appears to be to use an AC LVDT and separate modulator/demodulator. LVDTs are normally calibrated against mechanical or electronic micrometers, which are attached horizontally, or preferably vertically, to an LVDT mounting bracket. When using purely mechanical calibration systems care needs to be exercised to avoid errors due to slack, which can easily be produced by overshooting a desired displacement and then backing off. Basic calibration rigs of this type are adequate for establishing the sensitivity and non-linearity of electronic displacement devices. Higher quality rigs (some of which now use laser interferometry) can provide traceable calibration.

5.2.2 Volume change measurement

The traditional paraffin volume change gauge, designed by Professor A.W. Bishop and detailed in the first edition of Bishop and Henkel remains in use in many laboratories around the world. At the time of its development the main duties of a volume change gauge were to monitor changes in volume during the consolidation stage of effective stress triaxial testing, and in the triaxial consolidation test. In both cases the volume changes are generally quite large, and the equipment will perform adequately, although not with great precision, particularly as the inside of the measuring burette becomes dirty with increased use. If a 100cc burette is used, the typical resolution of the system is about 0.1cc, and the accuracy is presumably somewhat worse than this.

The introduction of electronic monitoring systems during the late 1960's and 1970's led to the development of electronic volume change gauges. Early versions were developed at Imperial College, and at the University of Surrey (Menzies, 1975). Both used Bellofram rolling diaphragms (BRDs) as the basis of measurement (Figure 12). The BRD has a small but significant rolling resistance which means that the supply (back) pressure may be somewhat different from the pressure supplied to the test cell. The Imperial College device also suffered from minor 'loss' of volume as a result of convolute stretching following a change in back pressure. Only a small volume change occurs as a backpressure change is applied to the gauge, because friction between the BRD and the apparatus walls restrains the BRD. During subsequent rolling the BRD extends or contracts (depending upon whether the back pressure

has been increased or reduced) as each new section of BRD passes through the convolute, resulting in progressively inaccurate volume change measurements.

External volume change measurements can be made more satisfactorily using a digital pressure controller (Menzies, 1988), which can detect volume changes as small as 0.5 mm^3 . The tubing connecting the cell to the pressure controller should be as stiff as practically possible. Stainless steel tubing is recommended.

In almost all tests the overall change in volume under changing effective stress is restrained to some extent by friction between the soil and rigid parts of the apparatus (e.g. triaxial end platens) that are in contact with it. In addition, in relatively coarse uniform grained soils (such as coarse single-sized sands) membrane penetration effects (see later) are a problem. The authors therefore prefer to measure both vertical and horizontal local strain in the mid-section of triaxial tests, and to compute the volumetric strain on the basis that, for a cross-anisotropic material

$$\epsilon_v = \epsilon_a + 2\epsilon_r$$

5.2.3 Pressure measurement

For a triaxial system measurement may be required of cell pressure, back pressure (at the base and/or top caps of a triaxial system), of the pore pressure at the base pedestal, and of the pore pressure at the mid-plane of the specimen.

As far as the authors are aware, the null indicator mercury pressure measurement system developed by Bishop and Henkel for base pore pressure measurement is no longer in use. Pressure transducers became widely available during the 1960s (Whitman et al., 1961) and, when mounted in a suitable de-airing block and charged with de-aired water, are easy to use, have very small compliance, and are virtually maintenance free. They also avoid the use of mercury, which is undesirable on health and safety grounds.

The first reported use of a pressure transducer in soil testing appeared, according to Bishop and Henkel (1962) in the 1950s (Plantema, 1953). Commercial laboratories currently routinely use pressure transducers to measure base pore pressure in effective stress testing. Measurement of pore pressures at the top cap is less satisfactory, because of the need to completely de-air the upper part of the system in order to obtain a rigid pore fluid and a rapid response time.

The response time of a pore pressure system is a function, amongst other things, of its compliance (Gibson, 1963). Soft systems require more flow of pore water, and therefore more time, before a change in pressure can be measured. Very soft measuring systems can reduce the excess pore pressure developed in the specimen, thus affecting the outcome of a test in terms of observed stress path, strength and compressibility. The compliance (in terms of volume take per unit increase of water pressure) of most modern pressure transducers is small compared with the potential volume change to air bubbles in ducts or around porous stones. But good results, and reasonable response times, can only be achieved if the system is free from gas bubbles. For ease of de-airing, pore pressure transducers should be mounted in a de-airing block, supplied from a ready source of high-quality de-aired water (see the section on 'Deairators' below). A de-airing block allows most air bubbles to be removed when the system is first assembled, and flushing between tests with high quality de-aired water ensures that any gas that finds its way into the system is dissolved and removed. The (careful) use of a high back pressure in saturated specimens helps stiffen the system and dissolve any residual gas bubbles.

During the shear stage of a triaxial test there will be a difference between the excess pore pressure developed in the mid plane of a triaxial specimen and that at the bottom of the specimen, to which the pressure transducer is connected (Bishop and Gibson, 1963). This difference arises as a result of platen friction, which produces different stress conditions near the ends of the specimen. When data are primarily required at or near the end of a test, then the rate of loading can be controlled so that pore pressures have time to equalise along the height of the specimen (see under Laboratory Test Methods and Interpretation below). When data are needed at all stages of the test, for example when following an effective stress path, mid-plane pore pressure measurement (see below) is advantageous.

In the UK the calibration of pressure transducers is generally carried out using a dead weight tester (for example, as manufactured by the DH-Budenberg Gauge Company), with calibrated weights traceable to national standards. With care, overall accuracy (from the instrument through the amplification and logging system) of better than 0.5 kPa can routinely be achieved over a 1000 kPa measurement range. Such a calibration system is an essential part of any laboratory aiming to produce high quality soil laboratory data.

5.2.4 Load measurement

As noted under the section on triaxial apparatus, early load measurement relied heavily on proving rings. The displacement of the ring was measured using a dial gauge, and later (in some cases) using

a displacement transducer. Proving rings have excellent linearity, but suffer from a number of practical disadvantages. They are quite bulky, cannot be placed inside the cell (to avoid ram friction), and have a limited dynamic range. Electronic load cells, on the other hand, cannot compete with proving rings in terms of combined non-linearity and hysteresis, but have greater dynamic range, can be calibrated and used over a number of switchable load ranges, and can be made submersible.

The use of submersible load cells, either mounted in the ram above the specimen, or under the pedestal, would seem essential for high quality testing. The typical load range for a submersible internal load cell used in routine UK soil testing is about 4000N to 5000N, whilst tests carried out on large diameter specimens of granular material may use load cells with ranges up to 25 kN. Given that a high-quality shear-web load transducer is likely to have combined non-linearity and hysteresis of about 0.1% of range, and that this is roughly equivalent to an error of 4 kPa in the deviatoric stress applied to a 38 mm diameter specimen, some care is required when testing soft clays or unbonded soils at low effective confining pressures. This problem can be overcome by using appropriate data-logging techniques (see below).

In the UK calibration of load cells is generally carried out using a dead weight pressure calibration test system coupled to a ram with an electrically-driven rotating bush. With care this system can produce compressive loads repeatable to better than 1N, which is considerably superior to the performance of a 4000N electrical internal load cell over full range. For smaller loads, and when load cells are to be used in extension, dead-weight hanger systems are used. Laboratory weights (used in this application and in others, for example oedometer testing) need to be regularly checked on suitable calibrated balances, and marked with their real weights. In the authors' experience laboratory weights have a long usage, and their mass is not always as stated.

5.3 *Ancillary equipment*

As Bishop and Henkel showed in their classic text, a considerable amount of ancillary equipment is needed to support laboratory, and particularly triaxial, testing systems. Over the years since their publication the selection of available equipment has if anything increased, and has changed out of all recognition. This section considers some of the more widely used devices that are available.

5.3.1 *Deairator*

A ready source of good-quality de-aired water is essential for triaxial testing. It is particularly useful when setting up new pore pressure systems, because trapped air will more readily dissolve into de-aired water. A convenient and practical design, termed the DeAirator was produced by Walter Nold and is in widespread use. It consists of a perspex cylinder that is filled with water and to which a vacuum is applied. The base of the cylinder contains an impeller driven through a magnetic drive coupling (to avoid leakage of air into the system). De-airing is accomplished by means vacuum application and agitation. Cavitation is produced behind the blades of the rotating impeller. The resulting agitation breaks up the liquid into a fine spray from which dissolved gasses are released and escape.

5.3.2 *Pressure sources*

Bishop and Henkel described in detail the self-compensating mercury constant pressure system that was used for many years in soil laboratory testing. This was followed in the 1980's by the use of precision air pressure regulators, coupled to air/water bladder interfaces that remain in use in many commercial laboratories today. Stepper-motor controlled precision air pressure regulators became common in the 1980's as sources of pressure for automated stress path testing. Servo-controlled pneumatic actuators are in fairly widespread use today.

The development of digital microprocessor controlled and computer controllable pressure/volume sources, such as those manufactured by GDS Instruments, has greatly increased the ease with which non-standard tests can be developed, for example for research. These units use a stepper motor drive and screw pump principle, coupled with on-board pressure measurement. A keypad and display allows constant pressure to be set locally whilst volume change is monitored, or other functions (e.g. linear time ramps and cyclic pressuring) to be used. The devices can also be easily programmed and logged through standard interfaces such as RS232 and IEE488 (Menzies, 1988).

5.3.3 *Fluid separation*

The pore water in soil is unlikely to match chemically that chosen by the investigator for the back pressure or cell pressure system (typically tap water or de-aired and de-ionised water), yet this seems rarely to be considered during testing. It can be particularly important when testing soils with saline or contaminated

pore fluid. Osmotic pressures cause pure water cell fluid to migrate through the membrane, potentially leading to large unwanted volume changes. This can be avoided by filling the cell with an immiscible fluid (e.g. silicone oil), and either providing an oil separator in the back pressure line, or completely filling the back pressure system with an immiscible fluid.

5.3.4 *Mid-plane pore pressure probes*

Because of the effects of platen friction, the stress state in a conventional triaxial specimen during loading is far from uniform e.g. Kirkpatrick and Belshaw (1968). Except in free-draining soils, differences in stress change result in differences in the magnitude of excess pore pressure. If, as is usual, effective stresses are to be inferred during testing, it is necessary either to test sufficiently slowly that drainage or pore pressure equalisation can occur at the mid-height of the specimen (see below), or to measure pore pressures in the centre of the specimen, remote from the effects of platen friction.

Two basic methods of mid-plane pore pressure measurement exist. In the first, which is suitable for tests on normally consolidated and lightly over-consolidated saturated clays, a miniature pressure transducer (e.g. a Druck PDCR81) is positioned on the surface of the specimen using a small pad of soft kaolin bedding, and sealed through the membrane using a rubber grommet and o-ring (e.g. Hight, 1982). Heavily over-consolidated clays will have sufficiently high suctions to de-saturate the porous stone on a PDCR81. If this occurs, for example during specimen set up, before the total stresses can be applied, then effective contact between the pore fluid and the sensor will be lost, and the instrument will fail to function as required. In this case a suction probe, such as that described by Ridley and Burland (1993), can be used. However, the proper de-airing of high-air-entry ceramic (the device described by Ridley and Burland uses a 15bar air entry ceramic) requires considerable care and time.

A simpler, more practical, but considerably more bulky alternative is to use the flushable mid-plane pore pressure probe described by Sodha (1974). In this device a purpose-built hollow ceramic (an air entry value of 250 kPa has been found adequate) is bonded to a stainless steel body (Figure 13). Two small-diameter stainless steel tubes are connected to the central cavity in the ceramic, and lead out of the cell to a valve block, one being connected to a source of high quality de-aired water. The ceramic is initially saturated by immersion under pressure in de-aired water. Satisfactory de-airing should lead to a response time of about 30 seconds if a change in cell pressure is applied to a probe sitting in a water-filled but otherwise empty cell.

During specimen set up it is mounted in a small hemispherical cavity cut into the side of the specimen, and seated using trimmings from the specimen, wetted up to produce a smooth soft paste. Initial contact with the specimen leads to cavitation of the pore water in the centre of the probe. As soon as a reasonable level of total stress (typically 200 kPa to 300 kPa) has been re-established at the start of the test the probe

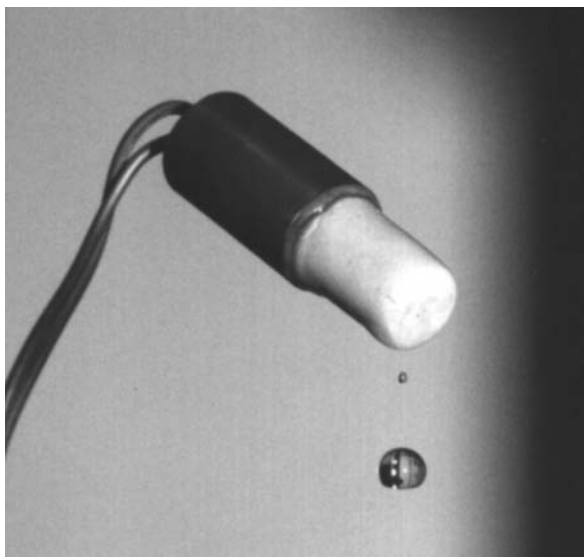


Figure 13. Mid-plane high-air-entry ceramic pore pressure probe (courtesy SGC Ltd.).

is flushed with de-aired water. A few small bubbles emerging during flushing is normally the sign that flushing has been successful. If so, further increases in cell pressure lead to measured B values very close to unity.

5.3.5 *Local and small axial strain measurement*

The conventional method of measuring axial strain in the triaxial test is now widely accepted as having significant limitations, particularly when stiffer soils are to be tested. At axial strain levels below about 0.5% the contributions of apparatus, load cell, porous stone, filter drain (Baldi et al., 1988) and (when used) lubricated end platen (Sarsby et al., 1982) to measured deformation are often greater than that of the specimen. Further errors are associated with bedding in of irregularities between the specimen and the loading platens (Daramola, 1978, Burland and Symes, 1982). Whilst the compliance of load cells and of loading frames can be compensated for with some justification, the compression of filter drains and the effects of bedding vary from test to test and are not very predictable. Research, largely carried out during the late 1970s and early 1980s, showed that much of the over prediction of ground deformation resulting from the use of laboratory data for stiff clays was attributable to the use of external strain measurement.

A further important development has been the appreciation that soils only exhibit linear stress-strain behaviour at very small strain levels. Not only is it necessary to conduct laboratory tests at relevant levels of effective stress, and along appropriate loading paths; stiffness must be measured at strain levels relevant to the field problem. Thus high quality triaxial tests for deformation-controlled problems (i.e. far from failure) must measure deformations both locally and at small strain levels. A great many principles have been used and systems devised for axial local small strain measurement in the triaxial apparatus, using for example:

- Submersible (Cuccovillo and Coop, 1997) and non-submersible (Ibraim and Di Benedetto, 2005) LVDTs.
- Electrolevel gauges (Burland and Symes, 1982, Jardine et al., 1984).
- Strain-gauged pendulums (Ackerley et al., 1987).
- Proximity transducers (Hird and Pierpoint, 1997).
- Hall Effect semiconductors (Clayton and Khatrush, 1986, Clayton et al., 1989).
- Strain gauged metal strips (LDTs) (Goto et al., 1991).

These devices are usually calibrated in the same way as external displacement sensors. Some (for example Hall Effect sensors) have noticeably inferior non-linearity when compared with commercial LVDTs. However it should be borne in mind that the displacements that are to be measured are ultimately used to plot some measure of stiffness (typically undrained secant or tangent Young's modulus) against the logarithm of axial strain. Resolution and variation in sensitivity over small measured distances control the effectiveness of a particular sensor (Clayton and Heymann, 2001). With care, and using an internal load cell with sufficient resolution, a number of these devices have been able to measure stiffnesses in the linear range, i.e. at axial strains of less than about $2 \times 10^{-3}\%$.

There are different views on the appropriate way to attach axial local strain gauges to the side of the specimen, with regard both to the length of the gauge relative to the total specimen length and to the method of fixing. As with other instrumentation, the stiffness of instrument cabling will be a concern if this restricts or imposes movements. Some experimentalists simply bond the fixing pads to the outside of the membrane using instant adhesive (Cyanoacrylate or Superglue), and use a gauge length that is as much as 80%–90% of the specimen length. This works perfectly well provided that the gauges are being used to overcome bedding effects, and that the gauge components hanging on the outside of the membrane do not rotate the membrane fixed to the top of the pads away from the specimen. If separation occurs then contact will be gradually re-established as cell pressure is applied, for example during a stress path test; the major component of measured local strain may then be due to pad rotation rather than axial straining of the specimen.

The first author prefers where possible to fix the gauge across the central third of the specimen, since in addition to eliminating bedding this minimises any effects due platen friction. In addition, he uses a combination of super-glue and pins to fix the pads to the specimen, since this not only ensures that the membrane under pads remains in contact with the specimen, but also provides a known gauge length from centre to centre of the pads. Where the pads are not fixed in this way there will be doubt as to the correct gauge length to be used in calculating local strain. Some experimentalists use the length between the pads, whilst others use the centre-to-centre distance. This can lead to significant differences in calculated strain.

The issues addressed above are clearly of less importance if gauges are light (as with the Japanese LDT) or non-contacting (as with proximity sensors).

5.3.6 *Radial strain measurement*

Radial strain measurement, using variants of Bishop and Henkel's 'lateral strain indicator', have been widely used and are still in use today, although with the mercury column replaced by electronic displacement measurement by LVDT or Hall Effect sensor. In the first author's opinion, the mechanical part of the original device works well, with low levels of slack and minimal resistance to movement.

Modern instrument designers would do well to follow the details of the spring-loaded hinge and pivoted radiused pads used by Bishop, or to replace the hinge with stainless steel spring wire (as in some Hall Effect devices). If the pad bearing area is too small and the resistance to opening and closing too large, relative to the stiffness of the soil being tested, then this type of gauge will under-register. A further design detail in need of careful consideration is the restraint caused by the typically-thick electronic cabling leading to a submersible LVDT (from this view point the flexible cabling used on Hall Effect sensors is preferable).

Radial callipers do not always deliver reliable results when used on coarser-grained soils in tests where effective stress is changed, because membrane penetration is associated with membrane thinning. A simple solution to this problem for tests on pluviated sand, devised by Bica (1991) is to bond thin radiused metal pads to the inside of the membrane at the location to be occupied by the lateral strain indicator. This does not, however, overcome bedding effects.

Other methods of lateral strain measurement include diametrically-opposed proximity sensors, and optical techniques such as PIV (see below).

5.3.7 *Imaging displacement measurement*

A number of 'optical' techniques have been used to determine patterns of displacement behaviour in or on soil samples. Classic examples, borrowed from soil centrifuge testing, include x-ray imaging of buried lead shot (e.g. Kirkpatrick and Belshaw, 1968) and the conventional imaging of surface markers.

In the testing of unsaturated soil, image processing techniques have been used to measure changes in the profile of specimens (Hoyos et al., 2005, Laloui et al., 2006) with considerable success. Particle Image Velocimetry (PIV) (White et al., 2001a, 2001b, 2003) is a promising technique offering the possibility of mapping displacements across the specimen, rather than only at the boundaries. Adapted from hydraulics, it measures displacements by comparing two digital photographs of the specimen taken at different times, corresponding to different levels of loading. For each pixel of the first photograph, cross-correlation and local interpolation are used to determine its new location in the second photograph.

The main advantages of using PIV, compared to traditional measurement techniques, are that:

1. PIV is non-intrusive, i.e. there is no need to attach instruments on the model/specimen or place markers in it.
2. It allows the measurement of displacements at arbitrary points of the specimen, rather than pre-determined ones. If needed, the displacement field for the whole model can be produced without the need for additional instrumentation. This is particularly useful in element tests, where PIV can reveal the extent of deformation in homogeneity or show the onset of localisation, before they become visible by naked eye.

PIV has been applied with success to measure deformations in centrifuge models, which deform under conditions of plane strain. The use of PIV to measure deformations during triaxial tests is still an open problem, since the shape of the specimen and the nature of the experimental setup introduce the following complications (Zervos, 2006):

- Due to the cylindrical shape of the specimen, the distance of different points around its circumference from the camera varies. Hence the scaling factor used to convert pixels to actual distance will not be constant for all points of a photograph.
- The specimen is wrapped in a membrane, which may obscure its texture. Additional texture must then be introduced on the membrane, e.g. by spraying.
- Refraction due to the Perspex cell and the water in it must be taken into account.

Despite these difficulties, triaxial experiments with PIV currently under way at the University of Southampton show considerable promise.

5.3.8 *Lubricated end platens*

The frictional end platens conventionally used in the triaxial apparatus result in stress and strain non-uniformity within the specimen, producing strain localisation and preventing samples being taken to the critical state, for example to study dilatancy. Lubricated platens or 'free ends' (Rowe and Barden, 1964, Bishop and Green, 1965, Barden and McDermott, 1965) use two or more layers of lubricated membranes in conjunction with specimens with an aspect ratio of about unity. Free-ends consist of enlarged end platens with a diameter greater than that of the specimen (e.g. 125 mm. for a 100 mm diameter $\times$ 100 mm high specimen) to accommodate the lateral movement; a disc of latex rubber membrane of size equal to the platen size attached to the platen loading face by a layer of high vacuum silicone grease usually referred to as lubricated membrane; and a small porous stone at the centre of each platen for saturation and/or pore pressure measurement. The effectiveness of this technique has been confirmed by Kirkpatrick and Belshaw (1968), and more recently by Frost (2003).

Probably because of a number of significant disadvantages, lubricated end platens have not come into use during conventional triaxial testing. Firstly, a special sliding type split sample former (Rowe & Barden, 1964) is required because of the lack of headroom. Secondly, drainage arrangements are made more difficult. Finally, when free ends are used, external deformation measurements cannot be relied upon, due to associated large and unpredictable bedding errors (Sarsby et al., 1982). Local instrumentation (and particularly the relatively light-weight large range Hall effect device described by Clayton et al., 1989) may be of help at smaller strain levels, but imaging systems are likely to be necessary to track strains over the entire strain range.

5.3.9 *Seismic velocity measurement*

The laboratory measurement of shear and compression wave velocities in rock samples using ultra sonic wave techniques is well established. A more recent development is the routine measurement of body wave velocity in soils using bimorphs or 'bender elements' (Shirley and Hampton, 1977, Schultheiss, 1981, Dyvik and Madhus, 1985). The bender element method has been used in a number of types of equipment including triaxial, oedometer, and simple shear and torsional shear apparatus, where transmitter and receiver have been installed in the top cap and base pedestal of the apparatus. In softer clays the bender elements can be pushed into the specimen. In stiff clays it is necessary to cut slots in the specimen and fill these with remoulded clay before inserting the bender elements. The problem of insertion into harder materials can be overcome by the use of piezoelectric shear plates (Brignoli et al., 1996). G_o has generally been obtained by assuming isotropy of stiffness, and the simple relationship

$$G_o = \rho V_s^2$$

where ρ is the density of the soil

V_s is the measured shear-wave velocity.

In a cross-anisotropic material the conventional (vertical) configuration measures V_{vh} . In a more recent development Pennington et al. (2001) installed bender elements through the sample membrane, allowing the determination of horizontal shear-wave velocities V_{hv} and V_{hh} at the mid height of the specimen. Measurements of G_{vh} and G_{hh} have been combined with measurements of static stiffnesses to define the full stiffness matrix for cross-anisotropic soils (Kuwano and Jardine, 1998, Gasparre et al., 2006).

It might be expected that the calculation of shear wave velocity from the travel time of a shear between two points in a soil specimen, created by flexing a bender element, should be straightforward. Two techniques have been used in bender element determinations of shear-wave velocity. The most common use time-of-flight of a single square or sinusoidal shear pulse, or a tone burst, to measure group velocity between a transmitter and a receiver. Phase sensitive techniques measure the phase lag of a continuous sinusoidal input over a range of frequencies.

In practice there are considerable uncertainties, stemming from inaccuracies in wave travel time due to both mechanical (e.g. coupling) and electrical effects, from uncertainty regarding the route taken by energy transmitted from transmitter to receiver and therefore the distance travelled, and from near-field effects. A number of techniques have been developed to reduce these uncertainties (Viggiani and Atkinson, 1995, Jovićić et al., 1996). Blewett et al. (2000) conclude that the combination of soil and bender element results in a frequency dependent response. Clayton et al. (2004) have suggested the use of co-planar side mounted bender element receivers, to measure the time interval of the same wave passing two similar receivers.

Given the uncertainties, and the current state of knowledge, the practical approach to using bender elements should be either:

- To use the results for comparative measurements of stiffness, or
- To ‘calibrate’ bender element results for particular soils and in particular apparatus and configuration against G_0 measured using other techniques, for example the resonant column apparatus

rather than regarding the method as capable of independently determining an absolute value of shear-wave velocity, and therefore of stiffness.

5.3.10 *Equipment for unsaturated soil*

The measurement of pressure and displacement in unsaturated soils has been considered in the sections above. In addition, a number of special pieces of equipment, external to the test cell, may be required. Both air and water pressure control are required, but when the axis translation technique is used these can be applied through the ports that are provided for their measurement.

Volume change measurement is also more complex than when testing saturated soils, because of the need to measure not only the volumes of air and water entering the cell, but also volume changes due to compression of the air phase in the soil, and of any air that diffuses through HAE porous stones into the water back pressure lines. Fredlund and Rahardjo (1993) state that greater accuracy of volume change measurement is required for unsaturated soils. A number of devices have been used for the measurement of air expelled from unsaturated specimens, including burette systems (Bishop and Henkel, 1962) and digital pressure controllers (Laudahn et al., 2005). Methods of dealing with diffused air have been described by Bishop and Henkel (1962 – a burette system), Bishop and Donald (1961 – the bubble pump and air trap) and Fredlund and Rahardjo (1993 – the diffused air volume indicator or DAVI).

Hoyos et al. (2005) discuss the various contributions that can be made by:

- Cell liquid measurement using conventional volume change gauges.
- The use of air and water gauges on the back pressure lines.
- Measurement of local strains and displacements on the specimen surface.

It would appear wise to adopt several different approaches in order that the various results can be compared with each other.

5.3.11 *Data acquisition and control systems*

The principles of good instrumentation and data acquisition have been understood for several decades. Since an advanced geotechnical test may be carried out over days or weeks, an important factor in soil testing is the long-term stability not only of the instrumentation, but also of any systems used to amplify, transmit and record or respond to data. Digital systems have largely overcome many of the early difficulties associated with data acquisition and control, and as a result automated soil laboratory testing apparatus is becoming widely available, both for routine testing and for special research testing. This is at once a tremendous advantage for geotechnical engineers, who no longer need learn of the complexities of (for example) standard computer interfaces, binary logic, and dual-slope analogue to digital (A/D) conversion, and at the same time a disadvantage, as they must deal with black box technology. Failure to understand the basic principles can lead to poor quality results, for example producing unacceptably low resolution.

In the context of A/D conversion, and the need to achieve resolution as well as accuracy for a range of soil behaviour, choice of instrument range becomes important. Some choices are relatively easy – for example pore pressure and cell pressure transducers will typically be required to operate over a 1000 kPa to 1500 kPa range. But internal load cells present more of a challenge. Firstly, the range of soil strength can be expected to be large, even over the limited range of effective stress typically used in strength testing. Secondly, the resolution and accuracy must be satisfactory at the very small deviatoric loads and strain levels at which stiffness may need to be measured. As an example, a natural soft clay such as the Bothkennar clay, with a very small strain modulus of 25 MPa will require accurate measurement of a deviatoric load of only 0.25 N at 0.001% axial strain, if tested as a 38 mm diameter specimen.

Several solutions are possible:

- A range of internal load cells could be purchased, so that one with an appropriate full-scale load could be installed for each test. This approach is not only expensive, but also requires regular disconnecting and re-commissioning of transducers, which makes detection of the onset of equipment failure more likely.

- One or two load cells can be used, but with higher dynamic range during analogue to digital (A/D) conversion. Although this method has potential, current systems do not seem to have a sufficient signal to noise ratio over the required dynamic range. It seems difficult to achieve more than 1 part in 13 to 14 bits of stability.
- A few load cells can be used, each being calibrated over a number of ranges. Combined non-linearity is generally found to reduce more or less linearly with the range over which the instrument is calibrated. The disadvantage of this approach is that higher resolution is needed over a smaller full-scale output, requiring higher amplification before A/D conversion.

For many commercial laboratories expenditure on specialist computer-controlled loading systems cannot be justified by the volume of work available. However, standard triaxial loading frames can readily be adapted to carry out stress path tests by adding a pneumatic rolling diaphragm actuator between the cross arm and the top of the ram. Because of the slow rate of loading necessary to ensure adequate drainage, and therefore that the desired stress path is being sufficiently closely followed, it is possible to break the desired stress paths down into small step changes (of one or two kPa each) in cell pressure and deviatoric stress, and apply these by hand. Manual control and data logging ensures close attention to the progress of the test, and in the first author's experience leads to fewer specimen losses than when automated, computer controlled, systems are used.

Commercial soil testing control systems have, in the past few years, become quite widely available. However, as they are typically coupled with bespoke black-box software, it can be difficult for the user to gain confidence in their performance. A simple (but perhaps rather expensive) solution preferred by the first author is to add 'redundant' instrumentation, independently logged and not linked to the feedback systems used to control the test, and use results from this for the analysis of test data. Test control can be carried out using digital pressure controllers, for example in the Bishop and Wesley stress path cell. In this configuration it is fairly straightforward to impose predetermined total or effective stress paths, under controlled strain levels (for an example, see Clayton, Xu and Bloodworth, in press). A more convenient alternative for commercial test houses is to use a standard RS232-controlled loading frame with digital controllers for cell pressure and back pressure, with feedback from an internal load cell being used as part of the control loop.

6 SAMPLING, SAMPLE DISTURBANCE AND SPECIMEN PREPARATION

Sample disturbance is sometimes considered in isolation from laboratory testing, but in order to obtain meaningful and worthwhile results it is necessary to match sample quality to the sophistication and purpose of the laboratory testing. Commercial tube sampling practice seems to be in a steady decline in the UK, yet engineers continue to specify complex and expensive laboratory testing designed to produce sophisticated parameters required for numerical modelling. It is doubtful if this can be justified.

From the perspective of laboratory testing, samples can be divided into three groups:

- Unrepresentative;
- Disturbed, and
- 'Practically' undisturbed.

It is not generally practical or indeed necessary to take the very highest quality samples. In many applications of commercial testing, the results are used in empirical or semi-empirical methods of computation, and it can be important to use the same test method as was used to derive the empiricism. When determining CamClay parameters, there is little point in paying for high-quality samples, and the same should be true when using the SHANSEP approach (Ladd and Foott, 1974).

6.1 *Representative and unrepresentative test results*

Samples may be unrepresentative either because they do not contain the full range of particle sizes that are present in the field or because they are not large enough or sufficiently undisturbed to contain sufficient fissuring or fabric to give results that are representative of the behaviour of the ground in the field. Low levels of disturbance will change the mechanical properties of a soil, but higher levels can distort, destroy or create fabric and through this also affect measured permeability.

Unrepresentative test results can also be obtained from inappropriate testing of otherwise adequate samples. For example, the coefficient of consolidation of a varved clay tested in a 19 mm high vertically draining oedometer will be quite different from the value controlling consolidation of an embankment

on the same material in the field. And the average undrained shear strength of 38 mm diameter triaxial specimens taken from 100 mm undisturbed samples of fissured London clay will be much higher than the value required to predict the short term stability of a slope in this material.

From the above it follows that the requirements to obtain a representative test result are more demanding than those required to produce a representative sample:

- The specimen should be sufficiently large so that the largest particle size fraction does not affect measurements of its behaviour.
- It should be large enough to contain representative discontinuities and fabric.
- Drainage in the test apparatus should be in the direction that will be dominant in the field situation.
- Mechanical disturbance should be sufficiently small that the parameters to be measured remain essentially unaffected, or that they can be recovered through re-establishment of effective stress levels or by normalisation.

6.2 *Phases and levels of disturbance (borehole, sampler, laboratory)*

Reviews of the mechanisms and effects of sampling disturbance can be found in a number of texts (e.g. Hight, 1986, Clayton et al., 1995, Clayton and Siddique, 1999, Hight, 2001) and will not be repeated here. Soil disturbance can occur, to a greater or lesser extent, during all the phases of the work, i.e. during

- Boring or drilling.
- Sampling.
- Transporting samples to the laboratory.
- Storage, and
- The initial stages of testing.

It often occurs primarily during the fieldwork phase of geotechnical investigation work. Most of the damage to the soil may be done before sampling starts, as a result of the boring or drilling method that is used. The engineer charged with obtaining a certain quality of sample generally has little control over the detailed operation except through an understanding of the processes that lead to disturbance, by specifying appropriately, by taking a keen interest in and supervising it, and by assessing the results after the event (see Sample Quality Evaluation, below). Alternatively, where possible, open excavations can be used to take samples whilst avoiding boring and drilling disturbance.

Sampling disturbance is now well understood, having been the subject of considerable research over a period of many decades. High-quality laboratory testing to assess the behaviour of natural soils sampled in different ways has played a major role in advancing our knowledge. For soft clays, experience with the Bothkennar clay has suggested that the highest quality samples are taken with a Sherbrooke device (Lefebvre and Poulin, 1979), which is essentially a down-borehole block sampler. The Laval sampler (La Rochelle et al., 1981) produced almost comparable results, and was considerably easier to deploy. Experience in London and elsewhere (for example, Hight and Jardine, 1993) suggest that the highest quality samples of stiff and stiff clays are obtained by using a wireline coring rig, with mud flush, and a triple-tube corebarrel. This technique is prone to cause swelling, particularly if the core is left in contact with thin non-bentonitic mud for any time. It has been suggested that the best samples are obtained for testing by removing the outer layer of the core immediately after recovery and before sealing.

Opportunities for soil disturbance do not cease once a sample has been taken from the ground. Soft soils need to be carefully supported to avoid inducing shear strains during transporting, extrusion and specimen preparation (Landva, 1964). Loose granular soils can be compacted by vibrations whilst in transit to the laboratory. Storage is important, since when done badly there can be significant drying of specimens (see for example Figure 6.13 of Clayton et. al., 1995), with attendant increases of mean effective stress, and presumably destructuring.

Further opportunities to change the behaviour of soft soil will occur during extrusion and sample trimming. Landva (1964) described the methods developed at the Norwegian Geotechnical Institute for preparing and mounting soft and sensitive clay specimens. More recently, Atkinson et al. (1992) have reported differences in the behaviour of Bothkennar clay specimens when trimmed in different ways. After preparation, the specimens were returned to their estimated in-situ anisotropic effective stress states, before being sheared undrained to failure. During shear there were significant differences in peak deviator stress, small-strain stiffness, and stress path. It appears from this, and the authors' experience, that a hand soil lathe and fine wire saw must be used when preparing specimens from larger samples.

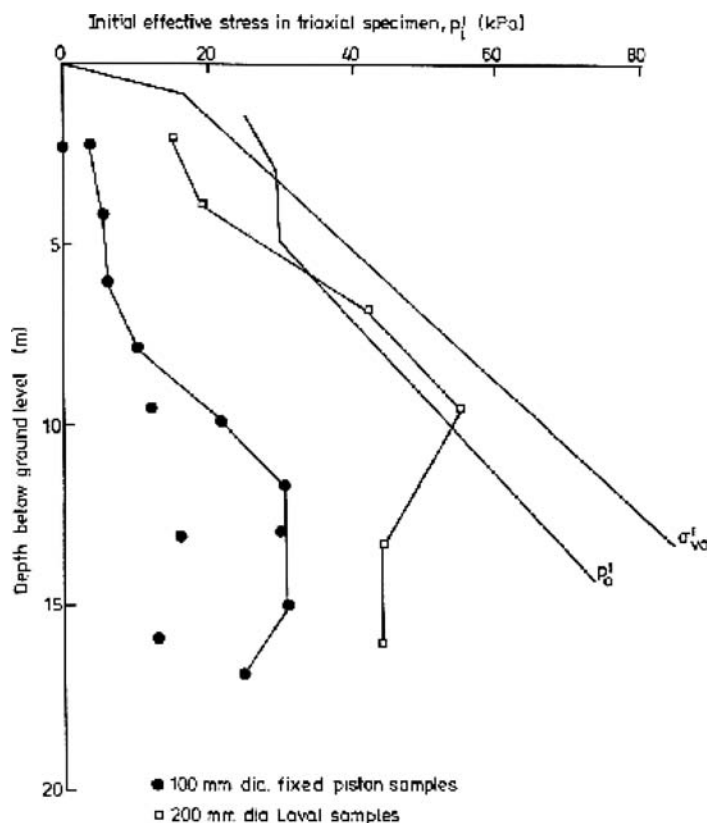


Figure 14. Initial effective stress in 100 mm piston and 200 mm Laval samples of Bothkennar clay ($PI = 18\text{--}22\%$).

It is essential the process of lathing, which can take some time, is carried out in a humid environment, so as to minimise evaporation.

6.3 Sample quality evaluation

If undisturbed samples are required, it is important that their quality is assessed before large amounts of time and money are expended on testing them. Various methods for assessing sample quality, i.e. the level of sample or specimen disturbance, have been proposed and these are discussed below.

6.3.1 Fabric inspection

Although of major importance for assessing some of the potential effects of sampling, in particular the risk of there having been water content redistribution between adjacent sand and clay layers, fabric inspection is not sufficient to determine the likely level of disturbance of the highest quality samples. Only gross distortion, for example in the distorted peripheral zone, can be seen, whereas relatively small strains cause yield and damage to a bonded structure. In addition, strain histories in the central zone of the sample involve unloading so that the maximum imposed strains cannot be deduced. Having said this, fabric inspection typically reveals the very poor quality of samples now routinely being sent for stress path testing in the UK.

6.3.2 Measurement of initial effective stress, p'_i

Comparison of the initial effective stress, p'_i , in samples taken from the ground and set up in the laboratory, with the value of p' that would be expected in perfect samples, has been advocated for many years as a means of assessing sample quality, e.g. Ladd and Lambe (1963). Data presented in Figure 14 shows how measurements of p'_i indicate a difference in quality between Laval and conventional UK piston

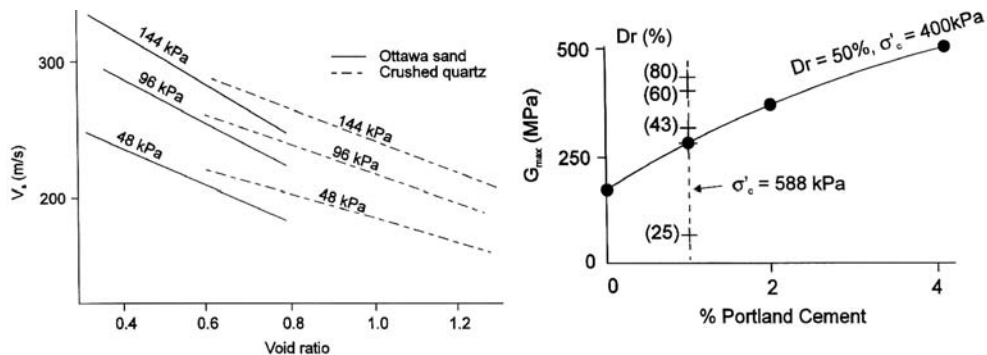


Figure 15(a) and (b). (a) Dependence of shear wave velocity, V_s , on void ratio (Data from Hardin and Richart, 1963). (b) Dependence of G_{max} on cementing and relative density (Acar and El Tahir, 1986, Saxena et al. 1988).

samples of Bothkennar Clay. Early measurements of p'_i can be obtained in the field or before laboratory testing on unconfined specimens using the Imperial College suction probe (Ridley and Burland, 1993). However, the measurement of p'_i alone is not sufficient as a complete indicator of sample quality, as it cannot indicate the amount of destructuring that has occurred.

6.3.3 Measurement of strains during reconsolidation

In reconsolidating samples to in situ stresses, the strains will depend on both the reduction in effective stress that has occurred as a result of sampling and the amount of destructuring. In this respect, they are a useful comparative measure of quality between samples. The absolute value of the strains will depend on the reconsolidation path followed and the soil compressibility. To take account of the latter, Lunne et al. (1997) have proposed expressing the volume strains in terms of $\Delta e/e_o$, where Δe is the change in void ratio and e_o the initial void ratio.

6.3.4 Comparison of field and laboratory measurements of shear wave velocity/dynamic shear modulus

The basis for adopting comparisons of shear wave velocity, V_s (or small-strain shear modulus, G_{max}), measured in situ and in the laboratory on the recovered sample, as a means of assessing the level of mechanical disturbance to structure is set out in Figure 15 for the case of sands. In sand, V_s depends on stress state and void ratio (Figure 15(a)), degree of cementing (Figure 15(b)), age (Figure 15(c)) and contact distribution. Figure 15(c) illustrates how a disturbance, in this case a twist, such as might be applied during rotary coring, changes V_s . The effects of ageing are removed, and, since the void ratio actually reduces, the reduction in V_s must be related to a change in contact distribution. A similar argument applies to clays in which the development of structure at constant stress is associated with an increase in V_s , while destructuring during sampling results in a reduction in V_s .

For the comparisons to be valid, the laboratory samples must be representative, in terms of fabric, discontinuities, etc. They must be restored to their in situ stress state, because of the dependence of V_s on stress state. Because of the dependence of V_s on void ratio, allowances must be made for changes in void ratio during reconsolidation to in situ stresses. Measurements of V_s should also be made with shear wave propagation in the same direction as in the field, with the same plane of polarisation and at a similar frequency.

An example of a comparison between in situ and laboratory shear wave velocity measurements in Bothkennar Clay is presented in Figure 16. In situ measurements of V_{vh} were made using the seismic cone. Laboratory measurements were made using bender elements, or the resonant column apparatus, on piston samples, taken with either a conventional sampling tube (30 degree cutting edge) or a modified tube (5 degree cutting edge), and reconsolidated to in-situ stresses. The improved performance of the modified tube compared to the conventional tube is confirmed by the shear wave velocity measurements. There is reasonable agreement between the in situ values and those measured on samples taken with the modified tube; values measured on samples taken with the conventional tube are lower, reflecting the structural damage which was known from other measurements to have occurred.

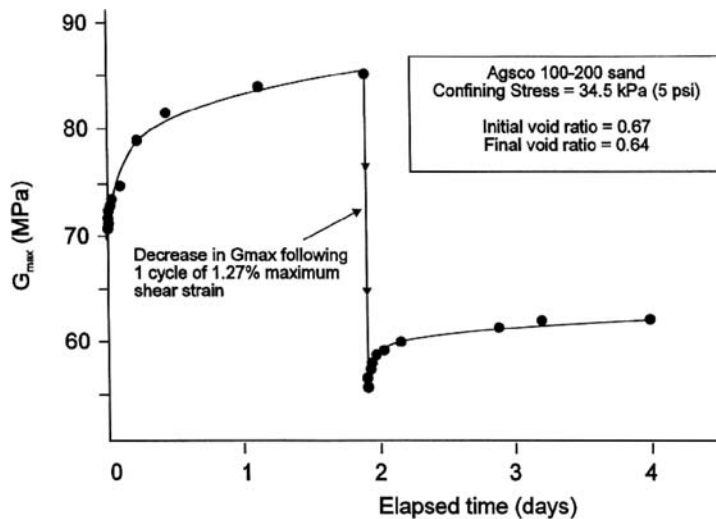


Figure 15(c). Effects of ageing and disturbance on G_{max} in sand (Thomann and Hryciw (1992)).

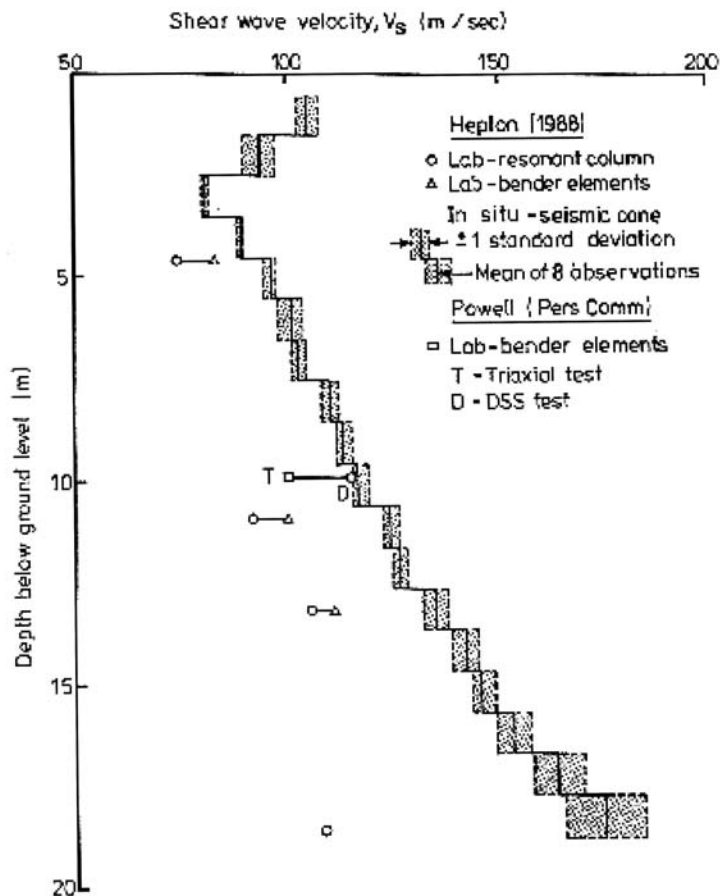


Figure 16. Comparison of laboratory and in-situ shear wave velocities. Bothkennar clay. 100 mm piston samplers (PI 18–22%).

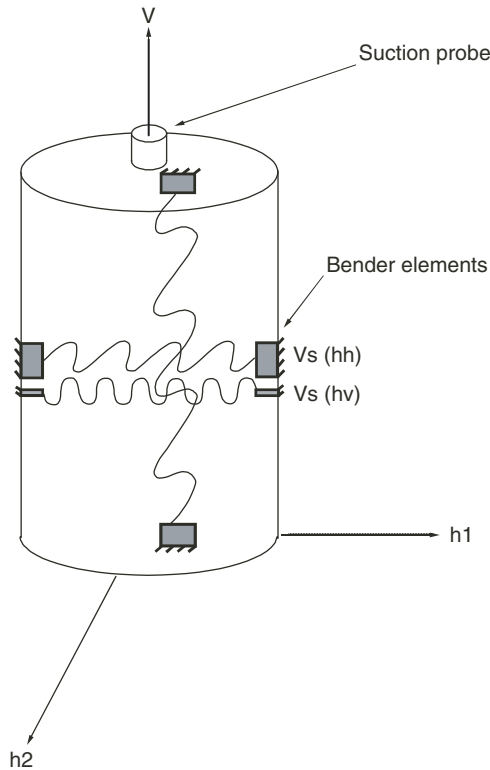


Figure 17. Bender element and suction probe arrangement for evaluation of sample quality (modified from Pennington et al., 1997).

6.3.5 On-site measurements of suction and shear wave velocity

All these methods are, however, relatively inefficient, as the critical information only becomes available after a few days testing time or when the test is complete. The evaluation may also be somewhat arbitrary since there is no clear yardstick against which to compare measurements such as volumetric strains during reconsolidation.

A method that can give an immediate indication of sample quality, prior to setting up the sample has been developed by combining two relatively recent developments, namely a portable bender element system, described by Pennington et al. (1997) and the Imperial College suction probe, described by Ridley and Burland (1993). The experimental set up is shown in Figure 17 and relies on the simultaneous measurement of suction and of the shear wave velocities, V_{vh} , V_{hv} and V_{hh} , in the unconfined (extruded) sample.

To isolate the effects of sampling on structure or fabric, allowances are made in the evaluation for changes in void ratio and stress state. Allowances for changes in void ratio are made by correcting the V_s data by a void ratio function. Allowances for differences in stress state are made by assuming that the exponents a and b in Roesler's equation

$$V_s = f \sigma_h^a \times \sigma_v^b$$

are equal to one. This allows comparison using suction raised to the power 2 for the unconfined samples and estimates of in situ stresses for the in situ measurements of V_s .

The method has been used to evaluate sample quality at a London Clay site, for which the data is shown in Figure 18, and at Bothkennar. The plot shows the variation with stress of in situ values of V_{vh} , V_{hv} and V_{hh} measured using downhole and crosshole seismic techniques. These are shown as parallel grey bands with V_{hh} at the top, followed by V_{hv} and then V_{vh} . These differences are a measure of the small strain anisotropy of London Clay. Equivalent data for reconstituted material is shown as parallel

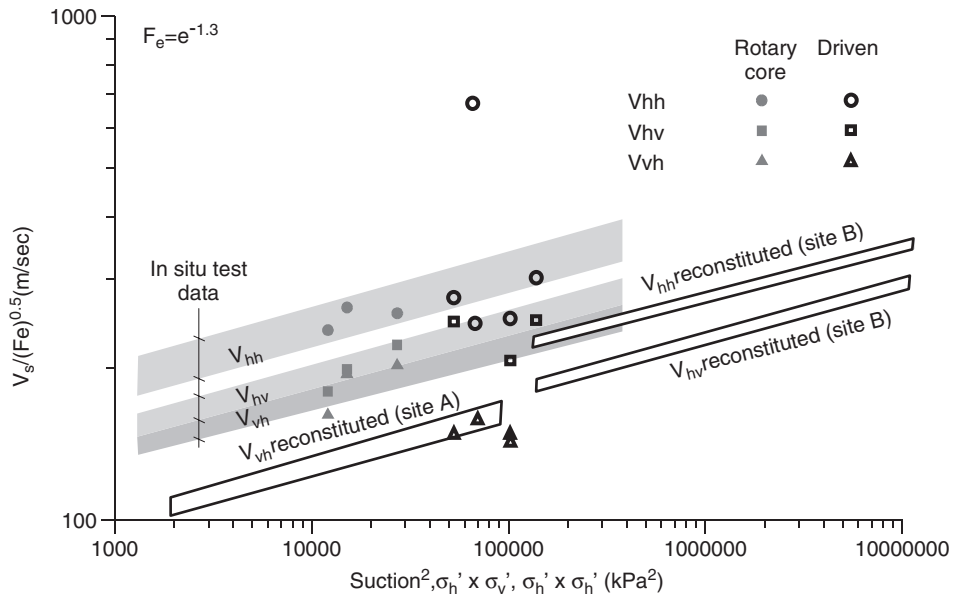


Figure 18. Evaluation of sample quality on a London Clay site using bender elements.

bands. The difference between the reconstituted and in situ data for a given wave type is a measure of structure in the clay, which, interestingly, maintains similar levels of anisotropy of small strain stiffness.

On this backdrop the values of V_{vh} , V_{hv} and V_{hh} , measured on three rotary-cored samples and three driven tube samples of London Clay, each pair being taken from similar depths has been superimposed. Good quality samples are identified when the shear wave velocities measured on them fall within the bands of the in situ data. The data for the rotary cored samples consistently falls within the appropriate bands, suggesting good quality samples. On the other hand, the data for the driven tube samples is more variable and values for V_{vh} fall well below the in situ data and even below that for the reconstituted material. These differences are obviously indicative of damage but demonstrate also how the small strain anisotropy is modified by sampling effects, probably by the creation of micro cracks having a preferred orientation.

7 USE AND INTERPRETATION OF LABORATORY TESTS

We have noted above the very considerable range of test equipment, instrumentation, etc. that now exists for testing soil. It is therefore often possible to obtain data on soil behaviour using different types of test apparatus and instrumentation, and experience suggests that these may (for good scientific reasons) yield significantly different results. A further complicating factor that needs to be recognised is that even on the same apparatus, using the same instrumentation, different test results may arise as a result of the use of different test methods. This section considers test methods for a range of soil tests. It is clear that the results of soil testing need to be considered in conjunction with data provided by other routes, for example in-situ testing, back analysis of field measurements, and desk studies.

7.1 Grouping and classification tests

Classification and index testing might be thought to be so simple that that there could hardly be any variability expected from different laboratories across the world. However, this is not so. Even for such routine tests, which are usually closely specified in national and international standards, variations in technique can have a large effect. The paragraphs below are by no means likely to be comprehensive, but have been drawn from our personal experience of commercial and research laboratory testing over a period of nearly four decades.

7.1.1 *Moisture content testing*

Though almost trivial in concept, the moisture content test is not carried out in the same way across the world. In principle the moisture content of a natural soil is considered to be the ratio of the mass of water to the mass of dry soil. These quantities can be determined by oven drying, but since clay particles contain both adsorbed and absorbed water, the latter being driven off over a range of temperature, the value of moisture content will depend to some extent on the temperature of the oven used to dry the soil, and on the types of clay mineral that it contains. The British Standard requires that the oven temperature should be between 105°C and 110°C. A different temperature may be required when preparing specimens for plasticity testing.

The determination of moisture content by oven drying requires not only that the specimen is heated but also that the specimen is held at that temperature for a sufficient time to drive off all moisture that can be removed at the specified temperature. Many test specifications demand that the soil is dried to constant weight, in principle requiring the operator to weigh the specimen repeatedly towards the end of the test. In the authors' experience of commercial testing this is never done. Specimens are held in the oven from the time that they are placed there until the following morning, when they are removed and weighed. Large specimens (for example necessary for coarse soils such as gravels) will not necessarily fully dry in this time (perhaps as little as 15 hours), particularly if the oven is comparatively full.

It is common in many UK laboratories to use numbered tins (with lids, which are intended to prevent moisture absorption during cooling – see below – but are rarely used) in which to dry soil during moisture content tests. Some laboratories use assumed weights for each tin, rather than weighing each one on every occasion that it is used. If the tin becomes dirty, or the lids are inadvertently switched, errors will occur. Further errors in moisture content can be produced if the soil is allowed to dry as it is being prepared for initial (wet) weighing, if it is weighed when still hot (as a result of air currents), or if the soil is left to cool in a humid environment (when it may absorb water). To avoid these last two problems some specifications therefore require specimens to be cooled in a desiccator before weighing.

7.1.2 *PSD and grain shape*

Because of the presence of cohesive fines, almost all coarse particle size distribution analysis is carried out using the wet sieving method. The method used is generally quite complex, requiring oven drying, removal (and hand cleaning with a wire brush) of particles coarser than 20 mm, separate sieving of the fraction coarser than 20 mm, riffing, dispersion with sodium hexametaphosphate, washing on a sieve to remove material finer than (in the BS procedure) 63 μ and finally, oven drying and sieving of the material retained on the fine sieve. Inadequate cleaning of the coarse fraction, incomplete dispersal (e.g. when the material contains clay-rich particles), poor washing, overloading of sieves, and insufficient sieving time can all lead to variations in the end result.

In the UK the method of sedimentation preferred by the British Standard uses the Andreason (fixed-depth) pipette, but in much of the rest of the world the hydrometer is still used. Given the fundamental objections to using Stokes' Law as a means of determining the size of fine soil particles, which are generally predominantly platy, the adoption of the more costly and demanding pipette method in the British Standard hardly seems justifiable.

Variations in particle shape, although recognised during description, cannot easily be measured. No standards exist.

7.1.3 *Plasticity testing*

Since only fine material is tested for plasticity, soil containing coarser material must be prepared for plasticity testing by sieving to remove coarse particles (the British Standard uses a 425 μ sieve). In practice variations occur as a result of differences in preparation technique, because when possible (i.e. in the absence of coarser particles) it is preferable to test the as-received material, without oven drying it. When a few coarse particles are present it may be possible to pick these out of the slurry prepared for testing, but it may be necessary to oven dry the material before sieving. The effect of drying temperature should strictly be assessed before a final method is defined, but in the authors' experience this is rarely one.

As noted above, the Liquid Limit is determined in many parts of the world using the Casagrande cup. In addition to issues relating to the hardness of the base, discussed above, the number of blows per minute (the British Standard specifies two drops per second) affects the rate of closure of the groove. An experienced operator can instinctively affect the results and achieve the repeatability required by standards by changing rate during the test.

Table 4. Results of a comparative testing programme to assess the repeatability of plasticity testing carried out in UK laboratories (Sherwood, 1970).

Plasticity Test	Soil type		
	Soil B	Soil G	Soil W
Plastic limit (%)			
Mean	18	25	25
Range	13–24	18–36	20–39
Standard deviation	2.4	3.2	3.1
Coefficient of variation	13.1	12.8	12.7
Liquid limit (%)			
(four point Casagrande method)			
Mean	34	69	67
Range	29–38	59–84	55–85
Standard deviation	2.4	5.2	5.3
Coefficient of variation	7.1	7.5	7.9

The high cost of plasticity testing in the UK (a liquid and plastic limit determination costs of the order of £25, as against £30 for an unconsolidated undrained (UU) triaxial test) seems to have dissuaded many commercial clients from carrying out the numbers of tests that are required for effective soil profiling. The 1-point method (Clayton and Jukes, 1978, Nagaraj and Jaryadeva, 1981, BS1377:1990 section 4.4) provides a cheaper alternative.

The Plastic Limit test requires only a glass plate and the hand and eye of the technician but clearly, since human hands are so variable and the detection of the crumbling point is subjective, it is unlikely that such a test will be highly reproducible. Table 4 shows the results of a trial carried out by the UK Transport and Road Research Laboratory in the 1970's, which tested inter-operator and inter-laboratory test variability of liquid and plastic limit determinations by sending the same material to over 40 laboratories. Most of the variability was thought by Sherwood to be attributable to operator technique. At the time the plastic limit test was widely used to determine the suitability of clay fill for use in highway embankment construction; following this study this usage ceased. It is clear that a better way of determining the Plastic Limit would be desirable, but the authors are not aware of any other, more suitable, methods.

7.1.4 Compaction and CBR tests

Because of the limited mould sizes that are used, most standards sensibly require removal of coarse particles (e.g. over 20 mm in the BS 2.5 kg and 4.5 kg rammer methods). However removal of coarse particles also changes the test result relative to that which would be obtained in a larger test.

Because of the sizes of the compaction moulds, it is necessary to restrict the types of soil that can be tested. BS1377 states that the 1 litre mould can only be used for soils passing the 20 mm sieve, and the CBR mould can only be used for soils passing 37.5 mm, with a limited percentage of the material coarser than 20 mm. Coarser materials are removed, and in some cases part of the finer fraction is substituted. Details of procedure such as these undoubtedly affect the results of the test, and probably make determinations to different national standards incompatible with each other.

In some laboratories and countries it is normal practice to use an automatic compaction machine, but in many it is not. It appears that the detail of the way in which the soil is compacted has an important impact on the result (e.g. Foster, 1962). Sherwood assessed the repeatability of compaction test results in 1970 and reported a maximum variation in optimum moisture content of 4–8%, with dry densities varying by up to 0.19 Mg/m³. The poor level of repeatability, coupled with the relatively long time required to get a result from laboratory testing, at least in comparison with the increasing output of earthmoving plant, has led to the adoption of a method specification (in contrast to compliance testing) for UK earthworks. Few compaction tests are now carried out for highways construction. In-situ density is increasingly determined using fast in-situ methods such as the nuclear density gauge.

7.2 Consolidation, compressibility and stiffness testing

The scientific background against which consolidation, compressibility and stiffness of soils are determined will have an important impact on decisions about the way testing is carried out. Based upon mechanics and the results of testing on pluviated, reconstituted and remoulded materials, the early view

was that soil compressibility was largely controlled by volumetric strains, and most affected by preconsolidation pressure (Casagrande, 1936). This idea continued well into the 1970s, but as the results of field monitoring and research laboratory testing became more plentiful and sophisticated it became clear that other factors were important.

It now seems to be widely accepted that almost all natural soils are significantly bonded, and that it is the progressive breakdown of structure during loading as well as changes in state that gives them their characteristic non-linear behaviour. Anisotropy of stiffness can be important, changing with increased strain, and needs to be taken into account. (Jardine, 2006). The relationship between stiffness and effective stress appears to change with strain level, being reported as a 0.5 power law at very small strains (within Jardine's (1992) Y_1 surface), and linearly proportional to mean effective stress at higher strains. At intermediate and large strain levels, the direction of the loading path has a significant effect on stiffness (for example, Clayton and Heymann, 2001). It has been suggested that recent stress history has some effect on stiffness (Atkinson et al. 1990), although this may be associated with creep effects.

7.2.1 Consolidation testing

BS1377:1990 Part 5 suggests that the oedometer test it describes is suitable for saturated or near-saturated soils. In the UK this is the standard method of obtaining compressibility and rate of consolidation for all cohesive soils. The specimen is mounted between saturated coarse porous stones, and placed in a cell filled with water. Loading is approximately doubled at each increment, and 'a typical test comprises four to six increments of loading, each held constant for 24 h'. Despite the disadvantages of the apparatus that have been described above, the oedometer test seems to be widely used worldwide in commercial laboratories to determine the compressibility of soils, and the rate at which they are expected to consolidate, for example under foundation or embankment loads. In stiff 'heavily over-consolidated' soils the values of both compressibility and rate of consolidation are very conservative, and therefore suitable for initial routine design. But for major projects representative values should be found in other ways, for example using triaxial testing.

In soft to firm clays oedometer testing appears more promising. As an example, Nash et al. (1992), give the results of an extensive suite of testing of the Bothkennar clay. Most of the tests they describe were carried out on high-quality Laval samples (La Rochelle et al., 1981), and three types of test (incremental loading (IL), constant rate of strain (CRS) and restricted flow (RF – Sills et al., 1986)) were used. Incremental load tests were not carried out in the routine way, but with many more loadings (20–30, generally applied daily) and smaller loading increments (around 10 kPa) around expected yield. Following Burland (1990) results were interpreted in terms of yield stress (classically termed 'pre-consolidation pressure'), yield stress ratio (classically termed 'over-consolidation ratio') as well as void ratio versus effective stress plots and the Compression Index, C_c . Little data of value appear to have been obtained pre yield. Different methods of determining the yield stress (i.e. Casagrande (1936), Janbu (1969), and Butterfield (1979)) produced significantly different results. Test results showed that the yield stress was somewhat dependent on the strain rate around yield, and that measured yield stress was quite variable (at best about $\pm 20\%$ of the mean, thought by the authors to be partly a function of natural variability).

It is well established that the effects of sample disturbance in soft clays are to increase pre-yield compressibility, reduce post-yield compressibility and to make the identification of the vertical yield stress more difficult. The tests carried out by Nash et al. from samples taken with the high quality Laval sampler did not give significantly different yield stress results from those on relatively poor quality specimens taken from an initial phase of piston sampling; this is the result of compensating factors.

Rates of compression can be obtained from maintained load consolidation tests, in the standard incrementally-loaded oedometer, in the hydraulic oedometer, in the triaxial dissipation test. Typically, these apparatus are used to determine the Coefficient of Consolidation (c_v) and, in combination with the Coefficient of Compressibility, m_v , the Coefficient of Permeability. These parameters can also be determined from CRS testing. In general, it is necessary to determine c_v for natural soils using the hydraulic oedometer or the triaxial dissipation test, since these allow larger specimens, with more representative fabric, to be tested. Where bedding fabric is pronounced, the hydraulic oedometer is to be preferred over the triaxial dissipation test; although filter drains can be used to encourage radial drainage, these are inefficient, because they are thin and have themselves a permeability that reduces under increasing effective stress. They cannot be relied upon to ensure zero excess pore pressure on the vertical boundary, and because their performance is unpredictable they should not be used.

The Coefficient of Secondary Compression, c_{α} , can be found from long-term tests either in the oedometer or under triaxial conditions. The extrapolation from laboratory to field is uncertain, since

issues such as the underlying mechanism, the effects of stress state and loading increment, and the impact of specimen height or (in the field) thickness of deposit, are uncertain. Terzaghi (1923) clearly viewed consolidation (which he termed 'hydrodynamic time lag') as a delay caused to the natural time-dependent compression of the soil skeleton by the inability of pore water to escape.

The influence of specimen thickness on predicted consolidation and secondary compression depends upon whether there is thought to be a unique strain-time relationship for the soil skeleton, or whether time dependent compression only starts at the end of primary consolidation, once excess pore pressures have dissipated (Ladd et al, 1977). The instability observed by Bishop and Lovenbury (1969), which is difficult to predict on a fundamental basis but may result from structural 'collapse', has also been observed in the field as excess pore pressure anomalies (Mitchell, 1986, Leroueil, 1988). The most common assumption is therefore that secondary compressive strain increases linearly with the logarithm of time, an idea originally proposed by Buisman (1936), and that it is independent of the specimen size. It is unlikely that this interpretation can be used with much confidence, although it probably yields conservative predictions of time dependent settlement.

c_α values determined in the laboratory are likely to be significantly affected by creep associated with bedding (e.g. when stiff clays are tested as thin specimens) and are certainly affected by the stress range over which they are determined. Nash et al. (1992), for the Bothkennar clay, showed very small amounts of creep until specimens were loaded to the in-situ vertical effective stress, after which creep rates increased, reaching a maximum at about the vertical yield stress, before decreasing somewhat. Creep rates clearly need to be assessed at stress levels relevant to the problem at hand, since the process is clearly associated with destructuring.

A further use of the oedometer is to determine swelling pressure, for example, see BS1377:1990 Part 5 section 4.3, and potential heave (for example Jennings and Knight, 1957, Williams, 1958 BS1377:1990 Part 5 section 4.4), or collapse upon wetting (Jennings and Burland, 1962, BS1377:1990 Part 5 section 4.5). The methods that have been used for such tests seem to differ quite considerably, and since in most cases the soil under test will not be saturated the results are probably best regarded as empirical indices. For example, Schreiner and Burland (1991) examined three 'standard' swell test procedures for estimating the amount of heave in the oedometer, using a hydraulic oedometer equipped with lateral total stress measurement and a high-air entry base ceramic. They noted that the soil underwent very different stress paths in the three methods, and as a result had different stress states at the end of the tests. They question the use of 'standard' procedures for design purposes.

7.2.2 *Triaxial testing of saturated soils*

Conventional triaxial testing is normally carried out using a number of well-established procedures,

- Unconsolidated undrained (UU) triaxial compression test without measurement of pore pressure.
- Isotropically consolidated undrained (CIU) triaxial compression test with pore pressure measurement.
- Isotropically consolidated drained triaxial (CID) triaxial compression test with volume change measurement.

Additional methods used on complex projects or for research include:

- Anisotropically consolidated undrained (CAU) triaxial compression test with pore pressure measurement.
- Anisotropically consolidated drained (CAD) triaxial compression test with volume change measurement.
- Ko consolidation (CKo) triaxial test.
- Undrained or drained triaxial extension tests, with pore pressure or volume change measurement.
- Drained stress path probes.
- etc.

Essentially, all the stages routinely applied to saturated or near saturated specimens can be broken down into one of the following:

- Saturation of apparatus.
- Measurement of initial effective stress.
- Isotropic consolidation (or swelling).
- Anisotropic consolidation (or swelling).
- Undrained deviatoric loading (in compression or extension).
- Drained deviatoric loading (in compression or extension).
- Rest periods.

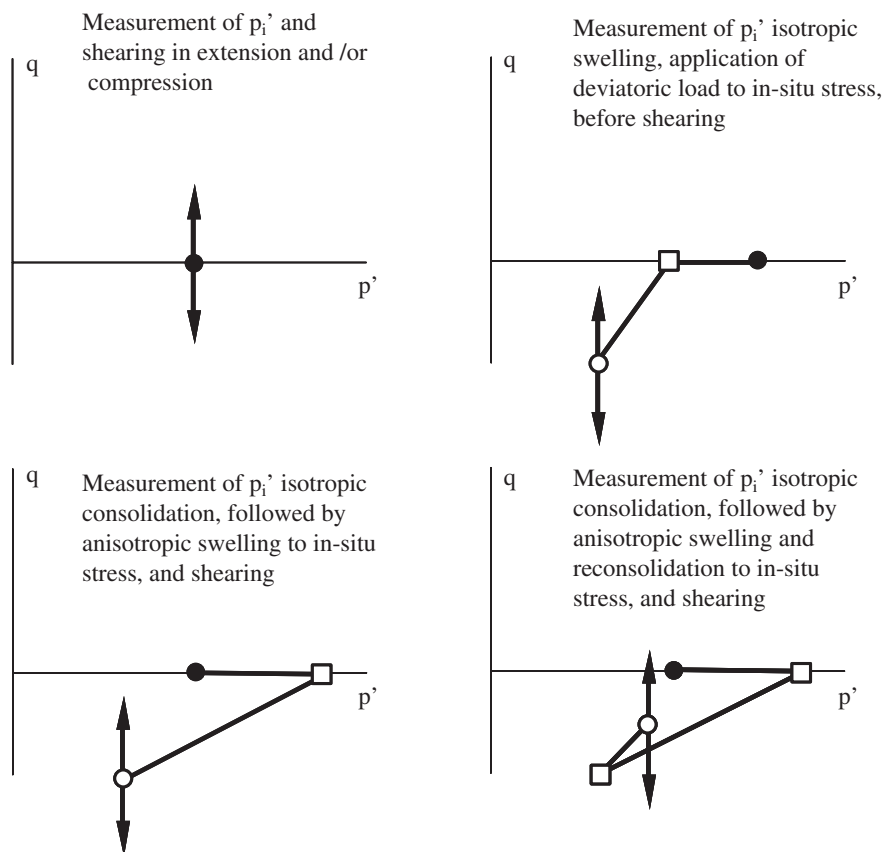


Figure 19. Example stress paths for undrained triaxial tests on heavily overconsolidated clay.

Jardine et al. (1991) give examples of triaxial test sequences used to determine relevant values of stiffness for basement construction. Figure 19 is based on these.

7.2.2.1 Saturation of apparatus

When testing saturated soils it is important that the apparatus (i.e. pore pressure ducting and tubing, and zones between the membrane, specimen, porous discs and platens) are also saturated, for otherwise

- The response of pedestal or top cap pressure transducers will be slowed, as a result of system compliance.
- Even after they settle, the pore pressures that are measured will not be the same as those on the mid plane.
- Two fluid pressures will exist, a pore air pressure and a pore water pressure, and there will be uncertainty as to what is being measured.
- Volume change measurements will be unreliable if back pressures are changed.

Two saturation procedures are described in BS1377:1990 Part 8. In the first, 'saturation by increments of cell pressure and back pressure', cell pressure and back pressure increments of between 50 kPa and 100 kPa are applied whilst maintaining the difference between them at between 5 kPa and 20 kPa. Saturation is considered complete when the B value exceeds 0.95 and the pore pressure remains constant for 12h or overnight. In the second procedure ('saturation at constant moisture content') the cell pressure is increased in stages, and the undrained pore pressure response is observed. Again, a B value of 0.95 is sought. UK saturation stages typically are held for about one day, although longer periods may be used with coarser-grained specimens. The specimen is enclosed in a latex membrane that has previously been soaked overnight, and is mounted between saturated (normally separate low-air-entry) porous discs, covered in water to minimise the amount of trapped air.

The wording of BS1377:1990 Part 8, Clause 5 makes it clear that for UK soils the techniques that are included are intended to be used to saturate the soil specimen itself. However Kuwano et al. (2001) rightly note that ‘all the details of the BS seem to presuppose saturated stiff clay’. The first technique, saturation by back pressure, requires long periods to be effective in clays (Black and Lee, 1973). In addition, high back pressures are required to saturate specimens with any significant initial lack of saturation. Lowe and Johnson’s (1960) calculations suggest, for example, that a back pressure of about 600 kPa is necessary to saturate a specimen with a 90% initial degree of saturation. Back-pressure saturation as described by the British Standard is therefore suitable for saturation of the apparatus rather than the specimen. Indeed, the sustained application of very low stresses induces unnecessary swelling that may lead to destructuring. The application of this technique to weakly-bonded and partially saturated soils is even more undesirable (Bressani and Vaughan, 1989).

7.2.2.2 Measurement of initial isotropic effective stress

The second ‘saturation’ procedure described by the British Standard can be applied to great effect on saturated soils to determine initial effective stress, but only with attention to detail. Recessed flushable high air entry porous stones should be bonded into the pedestal (and top cap if pore pressure is to be measured there). All ducts should be flushed with high-quality de-aired water, the membrane should be soaked and then surface dried, and the specimen slid onto the damp pedestal to minimise air trapping. Care should be taken to smooth the membrane up the side of the specimen, and remove as much air as possible before sealing the top cap in place. In stiff clays cell pressure increments should initially be held only for a short period, to allow pore pressure measurement, because suction in the specimen will draw water from the porous stones, and may cause cavitation in the de-airing chamber. Once it is estimated that the cell pressure has been raised sufficiently to induce a positive pore pressure in the specimen, cell pressure increments can be left until pore pressures stabilise. With additional increments of cell pressure the B value of a saturated specimen will approach unity, and the initial effective stress is found.

Extensive experience of using this technique in combination with flushable mid-plane pore pressure probes has shown it to be relatively easy to implement, and to give reliable results. The use of two independent HAE measuring systems, base and mid-plane, gives confidence in the results obtained.

Once the initial effective stress has been established a back pressure and cell pressure should be applied to give an equivalent isotropic effective stress, and the specimen should be allowed to settle. Experience with testing stiff clays suggests a minimum back pressure of 300 kPa. Considerably higher values may be needed in granular soils.

7.2.2.3 Isotropic consolidation or swelling

Isotropic consolidation or swelling stages are relatively easy to conduct. Unless a detailed knowledge of the volumetric compression v . effective stress relationship of the soil is required, they can be carried out by a single application of cell (or back) pressure change.

Volume change is best permitted by draining from the top platen, allowing pore pressure to be measured at the base, and on the mid-plane (as far as dissipation is concerned). However, since the time required for testing essentially controls the cost of the exercise (current UK commercial basic effective stress testing is charged at about £100/day) it may be preferable, where a precise knowledge of coefficient of consolidation is not required, to drain from both ends of the specimen. Mid-plane pore pressure measurement can still be achieved if a pore pressure probe is used. The speed of drainage can be further increased by using slotted or spiral side drains. Spiral drains are preferred for extension testing.

The progress of isotropic consolidation stages can be monitored by an external volume change gauge placed in the back pressure line, by base or mid-plane pore pressure measurement, or using local strain gauges. For routine testing the method of Bishop and Henkel is used. Volume change measured by a gauge in the back pressure line is plotted as a function of the square root of elapsed time. Measurements are continued until the volume change is thought to be essentially complete, and the straight line portion at the start of consolidation is extrapolated to the final volume change, to give what Bishop and Henkel term ‘ t_{100} ’ (actually this is $4.t_{50}$). Assuming the drainage boundaries to be perfect (i.e. that any filter drains have, despite being thin, initially having a finite permeability and compressing under effective stress, are fully effective and represent boundaries with zero excess pore pressure), Bishop and Henkel give equations for c_v which depend only on the boundary drainage conditions.

The effect of having side drains of limited permeability on the c_v value was examined by Bishop and Gibson (1963). The side drains were considered continuous (whereas they are slotted or spiral) and the permeability of the filter drain was assumed constant (whereas both thickness and permeability are both strongly effective stress dependent). Even so, they showed that the use of filter drains can lead to

large errors, particularly for more permeable soils. Nonetheless, the values obtained remain useful for estimating suitable rates of deviatoric loading (see below).

7.2.2.4 Anisotropic consolidation or swelling

In the triaxial apparatus, anisotropic consolidation or swelling stages require changes both in deviatoric load and effective all-round (cell) pressure. Such stages are typically used in order to control the stress path as the in-situ effective stress is re-established, to investigate stiffness along expected drained loading or unloading paths, or to carry out probes to determine the shape of the yield surface in the triaxial plane. In all these procedures the application of cell pressure and deviatoric load needs to be carried out slowly, in order to make sure that excess pore pressures do not lead to significant deviation from the desired effective stress path.

Although in theory it may be possible to predict a suitable rate of loading from the rate of drainage during isotropic consolidation, in practice it is better to control the rate of stress application by setting limits on excess pore pressure measured at the mid plane, which in practice can be best done using mid-plane pore pressure measurement sensors. Excess pore pressure limits can be set in relation to either the mean effective stress or the vertical effective stress, as deemed appropriate. A value of 5% has been used in the UK (Jardine et al., 1991).

Anisotropic consolidation stages are extremely time consuming, and therefore should only be included where they are justified on technical grounds. Some simplifications to stress paths may be possible. For example, it may be reasonable to approach anisotropic in-situ stress conditions by first applying a single isotropic consolidation (or swelling) stage, and then following this with a stage where only the deviatoric stress is varied.

7.2.2.5 Undrained deviatoric loading (in compression or extension)

Routine determinations of the effective strength parameters of clays are commonly carried out using the conventional consolidated undrained triaxial compression test, described by Bishop and Henkel (1962) and standardised (amongst other places) in BS1377:1990 Part 8 (see below). However, this type of test is also widely used (in conjunction with a stage to determine the initial effective stress of the specimen, and if sampling has altered it significantly, an isotropic consolidation or swelling stage) in 'routine' determinations of small-strain stiffness.

Whereas in routine triaxial testing the machine rate of displacement is set according to a rate calculated on the basis of the coefficient of consolidation determined from an isotropic consolidation stage, small strain stiffness measurement is invariably carried out using both local strain instrumentation and mid-plane pore pressure measurement, at a slow but standard rate of loading. In the UK it is typical to use a machine rate of displacement equivalent to an axial strain rate of 4%–5% per day when carrying out tests of this type on stiff clays. This gives reasonable levels of equalisation between base and mid-plane pore pressure measurement, and allows sufficient small-strain data to be taken in the early stages of the test.

However, following many years of apparently successful use of this technique in the UK (e.g. Jardine et al., 1991) recent research at University College, London (Baudet, pers. com.), discussed below, has suggested that the S-shaped curve so produced may also partly be a function of strain rate.

7.2.2.6 Drained deviatoric loading (in compression or extension)

Bishop and Henkel (1962) note that although suitable rates of loading can be found by trial and error, it is convenient to be able to calculate suitable values from the data obtained during the consolidation stage(s) of a test. For a drained test the applied rate of loading needs to be calculated on the basis of estimates of strain and time at the first point at which measurements are to be considered valid. For example, in a routine isotropically consolidated drained triaxial compression test, the applied rate of loading must be calculated using the strain at which the soil is expected to fail, which is difficult to estimate without experience and should be checked at the end of the test, and the calculated minimum time to failure. For tests where more data are required, the loading rate must be set on the basis of the strain after which data are required.

Gibson and Henkel (1954) and Bishop and Gibson (1963) showed that the average degree of consolidation during a drained test can be expressed as:

$$\bar{U}_f = 1 - \frac{h^2}{\eta c_v t} \quad \text{or} \quad t = \eta c_v \frac{(1 - \bar{U}_f)}{h^2}$$

where $2h$ = height of specimen
 c_v = coefficient of consolidation
 t = time
 η = a factor depending on drainage conditions at the sample boundaries. Values of η are given by Bishop and Henkel (1962) in their Table 6.

Bishop and Henkel show that a degree of dissipation of 95% is sufficient to give negligible testing error as a result of the presence of significant excess pore pressures. t is the time after which test measurements become valid. In other words, for a routine effective strength (CID) test where data are required to be accurate only at failure, t is the time to failure and the applied rate of machine displacement is calculated from this and the failure strain. This approach is advocated by BS1377:1990 (Part 8, section 6).

For tests requiring small-strain stiffness measurement, the rate of drained deviatoric loading should be controlled to keep mid-plane excess pore pressures within the specified bounds, as described under the section on 'Anisotropic consolidation or swelling' above.

7.2.2.7 Rest periods

Current thinking in the UK is that any test carried out to determine stiffness must include rest periods before those loading stages intended to produce small-strain stiffness data (compare, for example, data given by Atkinson et al. (1990) with those of Clayton and Heymann (2001)). Creep strains occurring as a result of previous loading stages cannot be distinguished from strains arising from the current load stage, and the sum of these can only be attributed to the current loading path. The effects of recent history reported by Atkinson et al. (1990) have been interpreted as an artefact produced by inadequate rest periods.

The most reliable way to produce small-strain stiffness data is to use undrained compression or extension stages loaded at a constant machine rate of strain. Once the machine rate of strain has been decided upon (e.g. 5%/day) the creep rate in the preceding rest period can be identified. Due to bedding, the actual rate of strain of the specimen will, in the early stages of the test, be much smaller (perhaps 1/10th) than of the machine rate of strain, and the creep rate needs to be small in order to take this into account. If control of strain rate based on local measurements is not feasible a value of 1% of the proposed machine rate of strain is suggested.

However, data recently produced by Kataoka Sorensen (2006) apparently does not support current practice. Results of tests carried out on undisturbed London Clay specimens suggest that the S-shape stiffness curve used for characterising a soil will to some extent be a feature of the test procedure rather than the soil behaviour, with measured stiffness being affected by the creep that occurs at intermediate and larger strain levels.

7.2.3 Dynamic, cyclic and torsional triaxial testing

The techniques described in this section (the resonant column test, and cyclic axial and torsional triaxial tests), are used to determine dynamic or cyclic stiffness. Cyclic triaxial and cyclic torsional tests are usually carried out in Japan to determine the deformation properties of soils. In the UK cyclic triaxial tests are unusual, sometimes being carried out for repeated loading problems such as rail and road formation testing. Only a few cyclic torsional apparatuses are thought to exist in the UK (e.g. at Imperial College, London). Cyclic simple shear testing is also used in some specialist applications, e.g. to obtain cyclic data for offshore foundation analyses.

The resonant column tests soil at very small to small strain levels, and relatively low frequencies, and therefore provides data that can give a benchmark of stiffness for comparison with other tests, can be used to estimate seismic stiffness properties, and for the determination of soil properties for ground response analysis under earthquake loading. Because of the need to ensure free vibration, local measurement is not used on the resonant column. This does not appear to be a problem when testing relatively soft materials or reconstituted sands, because the apparatus uses rough platens to ensure fixity. Calibration tests for much stiffer material (methane hydrate – with $G_{\max}$ up to 5000 MPa, Clayton et al., 2005) have shown the mass polar moment of inertia of the Stokoe apparatus to vary with calibration bar stiffness, although the reasons remain uncertain.

The cyclic triaxial apparatus is typically used to determine the resilient modulus and plastic strains that may be expected to occur under repeated loading. Kuwano et al. (2001) comment that the test as specified in ASTM D3999 does not insist on internal (and therefore local) strain measurement. However, bedding will affect even the resilient modulus in stiffer materials, and in the authors' experience it is essential that it is used. The problems of bedding appear to be reduced when using monotonic or cyclic torsional shear.

The work of TC-29, under the leadership of Professor Tatsuoka during the period 1994–2001 (Tatsuoka et al., 2001), produced a number of important conclusions, partly based on round-robin testing in Italy (Cavallaro et al., 2001) and Japan (Yamashita et al., 2001) and in general using resonant column, cyclic triaxial and cyclic torsional testing. These can be summarised as follows:

- Dynamic and static moduli measured in different ways should be the same provided that they are measured under the same stress and void ratio conditions, at strains less than 0.001%, since rate effects at this strain level are small.
- The stress-strain behaviour of geomaterials is highly non-linear, and dependent on effective stress, as well as void ratio. This must be taken into account when interpreting test results.
- Consistent and reproducible results can be obtained in different laboratories when following the same procedures.
- The use of local strain measurement is essential when testing stiffer materials.
- G_0 values need to be corrected both for stress level and void ratio. Sampling disturbance may reduce G_0 . In the Italian round-robin test programme G_0 values measured in the laboratory varied from about 50%–100% of those measured in the field using the cross-hole technique. In the case of the Japanese Sagami-hara soft rock, laboratory values were very similar to field values.
- Stiffness degradation data obtained from different tests and different laboratories are quite reproducible.
- Different methods of setting up specimens (e.g. the so-called ‘dry-setting’ and ‘wet-setting’ methods, Lo Presti et al., 1998) can have a large effect on stiffness both at small and large strains. Measures to minimise swelling during specimen set up are necessary.

7.2.4 *Advanced testing for stiffness*

Based on local measurement of displacements of hollow cylinder samples and calculations of average strains, stiffness data can be derived for different directions of the major principal stress direction during shear and during consolidation. Investigations of the anisotropic stiffness of silt are reported by Zdravkovic and Jardine (1997) while Gasparre et al. (2006) describe the results of HCA tests on block samples of London Clay, aimed at defining the anisotropic stiffness of the clay. In the tests on London Clay the axial and torsional shear strains were measured with an enhanced-electrolevel system, while radial and circumferential strains were calculated from the outer and inner diameter changes monitored with a set of three proximity transducers, and a laterally mounted LVDT, respectively. Taking multiple readings and using an averaging routine allowed strains to be resolved down to around 0.0003%.

The fact that local instrumentation can be deployed in an HCA gives it a major advantage over apparatus such as the Directional Shear Cell, which relies on observations of the displacement of markers in the specimen to derive strains.

7.3 *Strength tests*

For practical purposes, undrained shear strength (s_u) is required for the solution of ‘short-term’, ‘end-of-construction’ problems, whilst effective stress strength parameters (c' , ϕ' , c'_r , ϕ'_r) are required for problems where stability is critical in the long term. All failure envelopes are curved, so that effective stress strength parameters need to be assessed at stress levels relevant to the field situation to which they are to be applied. For example, slope failures generally occur at relatively shallow depths, so vertical stress levels adopted during testing must be suitably small. Failure to do so can produce large cohesion intercepts that will grossly overestimate the stability of a slope.

Strength testing needs to be carried out within the now well established framework for natural and reconstituted cohesive soils (for example, see Burland, 1990, Leroueil and Vaughan, 1990, Jardine, 1992, Leroueil and Hight, 2002, etc.). Natural soil exists within an approximately constant volume Local Bounding Surface that can be mapped using a combination of undrained and drained stress excursions from the in-situ stress state. As load is applied the soil behaviour is progressively modified. After a stress change of as little as 2 kPa linear stress-strain behaviour ceases, but strains and stiffness continue to be recoverable. After larger stress excursions irrecoverable strains start to accumulate, and eventually large scale yielding occurs as the Local Bounding Surface is approached. Drained stress paths that cross this surface lead to destructuring, and significant changes in strength.

It is sometimes helpful to carry out additional tests so that the behaviour observed in what we have termed ‘practically undisturbed’ specimens can be compared with reference states. Examples of these types of frameworks include the use of the ‘Critical State’ framework (Schofield and Wroth, 1958), and ‘intrinsic’ properties (Burland, 1990).

7.3.1 Triaxial testing

The use of the triaxial apparatus has a number of advantages and disadvantages. As noted above, specimens are relatively easy to prepare, and associated ranges of test techniques and instruments have become available, and to some extent standardised. With regard to determination of strength, effective stresses can be applied or inferred, and measurements can be made both in compression and extension, at various stress levels relative to the vertical effective yield stress of the soil, and at peak and (in special tests) under residual conditions. Disadvantages include the fact that triaxial testing is carried out under special stress conditions, where two of the principal stresses are equal. In contrast, many geotechnical problems occur under conditions of plane strain and with principal stress directions rotated from the vertical. While strengths in plane strain compression tend to be higher than those in triaxial compression, i.e. with the major principal stress direction vertical, compression strengths are often higher than strengths for other directions of the major principal stress.

7.3.1.1 Disturbance during re-application of effective stresses

It is easy to do damage to the structure of a low-strength, low OCR soil specimen during the early stages of a test. Drained isotropic and K_o stress paths cause comprehensive destructuring of clays as they cross the Local Bounding Surface, leading to very significant changes in soil properties as the material moves towards the large strain outer State Boundary Surface. Subsequent stress-strain loading produces much-reduced brittleness, and contractive behaviour, in contrast to what is seen in a structured soil. Care should be taken to re-apply appropriate levels of effective stress at the start of testing, and in a manner that does not damage bonding structure.

At slightly higher over-consolidation ratios similar behaviour occurs, but destructuring appears more progressive and observed undrained stress paths are sensitive to disturbance, for example caused by tube sampling and specimen preparation (see above), which will significantly affect peak strength. For these types of clay there is some evidence that the stress path used to reach the initial in-situ stress point at the start of a triaxial test can also affect pore pressure generation, and hence the stress path in undrained triaxial compression tests (Hight et al., 1992).

7.3.1.2 Limitations after failure

When loaded in triaxial extension or compression at the relatively small mean effective stress levels associated with heavily over-consolidated clays, most soils show approximately vertical stress paths in t - s' space. Strain localisation occurs at or close to maximum deviator stress, which is seen to occur above the Critical State Line characterising reconstituted material. Beyond the onset of strain localisation test results cannot be accurately interpreted in terms of stress and strain.

7.3.1.3 Pore pressure equalisation and times to failure

Routine consolidated undrained (CIU) triaxial testing with pore pressure measurement measures pore pressure at the base of the specimen, remote from the central section where failure should occur. Deviatoric loading causes variations in stress increase within the specimen (as discussed above) that in an undrained test lead to differences in excess pore pressure. Testing must be slow enough to allow adequate equalisation of pore pressure between the base and centre of the specimen. Gibson (in Bishop and Henkel, 1962, Appendix 6) produced theoretical time factors for vertical flow only and for vertical and radial flow, with an infinitely permeable boundary over the whole specimen surface. From this, the time for 95% equalization is:

$$t = \frac{T h^2}{c_v}$$

where $2h$ is the height of the specimen

c_v is the coefficient of consolidation, obtained from a previous isotropic consolidation stage

T is the time factor, 1.67 for 95% equalisation without side drains, and 0.071 for 95% equalisation under the assumption of an infinitely permeable layer over the entire soil surface.

If pore pressures are not measured at the centre of the specimen, these factors must be used to define the time after which the first valid readings of pore pressure can be taken. For routine testing, the axial strain at failure is estimated based upon experience, and the machine rate of strain is calculated from this

and the minimum time to failure obtained from the above equation. Given the uncertainties in predicting failure strain, and the fact that filter drains are neither continuous nor infinitely permeable, caution needs to be exercised in the use of this approach.

7.3.1.4 Membrane and filter drain effects

At small strains, on large-diameter specimens, and for stiff soils, the additional axial resistance provided by a membrane is generally insignificant. However, at failure and on soft soils the combined effect of membrane and filter drain can be considerable (Henkel and Gilbert, 1952, Bishop and Henkel, 1962 Appendix 1, and need to be taken into account when calculating deviatoric stress. Corrections such as those given in BS1377:1990 Part 8, Figure 4 and Table 2 although partly based upon Bishop and Henkel's work seem somewhat arbitrary, but indicate that the effect upon deviatoric stress on a 38 mm diameter sample may be up to 12 kPa. The methods given by La Rochelle et al. (1988) and Leroueil et al. (1988) are based on sound principles and measurement, and appear much more suited to the task.

7.3.1.5 Normalisation

As an alternative to attempting to measure the undisturbed behaviour of soil, the SHANSEP method (Stress History and Normalized Soil Engineering Properties – Ladd and Foott, 1974) was developed with the aim of removing some of the effects of sampling disturbance. Ko anisotropically consolidated triaxial tests take specimens to vertical stress levels beyond the in situ pre-consolidation pressure, and the strength results are normalised by vertical effective stress and OCR. Although this technique must destructure the clay that is tested, it appears to produce reasonable results for those soils that do not have significant bonding structure. For example Jardine et al. (2003) suggest that even for the soft lightly over-consolidated Thames Estuary Clay, the loss in strength was only of the order of 5–10%, although there was a more significant reduction in stiffness. This technique therefore offers a practical means of obtaining more realistic strength properties when high quality undisturbed samples cannot be obtained.

7.3.2 Direct and simple shear testing

Because of the lack of drainage control, the direct shear test is generally used for drained testing. In clays it is necessary to carry the test out sufficiently slowly to ensure that excess pore pressures have been able to dissipate by the time failure is reached. The theory presented by Gibson and Henkel (1954) can be used. With a degree of excess pore pressure of 95%, their test results suggest that a strength within one or two percent of the fully drained strength will be developed. Drained shear box tests are no longer routinely used in the UK to determine peak effective strength parameters; the isotropically consolidated undrained (CIU) triaxial test is preferred.

Undrained strengths of soft clays measured in constant volume simple shear tests using the NGI simple shear apparatus have been found by experience to provide a reasonable average of the triaxial and extension strengths on the same sample, and to provide a good estimate of the average strength mobilised during failure of an embankment on the soft clay. On this basis the test has become popular in certain parts of the world, in particular in Eastern USA and Scandinavia. As principal stresses are rotated in the test, the strengths measured contribute, together with those from undrained triaxial compression and extension tests, to an appreciation of the likely level of undrained strength anisotropy.

7.3.3 Testing for residual strength

Although the direct shear test concept at first appears very simple, the loading and distortions applied to the soil are complex. Study of the development of failure within clay specimens (Morgenstern and Tchalenko, 1967) showed that failure initiated at stress concentrations at the ends of the box, producing a series of inclined Riedel shears that eventually coalesced to produce a wavy sub-horizontal failure surface. The uneven geometry of the confined shear surface leads to high values of residual strength, and these continue even if many reversals are applied in an attempt to smooth the surface and align the clay particles. The determination of both peak and residual strengths in a 'stress-reversal shear box test' is therefore not recommended.

The shear box can, however, give good estimates of residual strength parameters if drained tests are carried out on samples containing shear surfaces recovered, for example, from exploratory pits, provided that the shear surface is carefully aligned with the split in the box, and the normal effective stress levels match those in the field (see below). A number of passes are made, with the specimen unloaded during reversal to avoid disrupting the shear surface. Skempton (1985) compared the residual angles of friction derived from direct shear tests made at slow rates (<0.01 mm/minute, to avoid generating excess pore pressures) with values obtained from back analysis, and found that shear box tests on residual shear surfaces gave,

on average, slightly high residual angles of friction, by about 0.5° . From the scatter of data he concluded that 'even in the almost ideal conditions of these case records, where pore pressures are known with reasonable certainty and problems such as the effects of progressive failure are absent, stability analysis and laboratory tests cannot be expected to yield results with an accuracy better than about $\pm 10\%$ '.

The cut-plane shear box test (Skempton and Petley, 1967) provided a logical response to determining residual strength, in the light of Morgenstern and Tchalenko's findings, and the normal difficulty of obtaining samples containing shear surfaces. For a cut-plane shear box test the specimen is first swelled to low effective stress, then reconsolidated to the required vertical effective stress level. A planar shear surface is then produced by slightly separating the two halves of the box, cutting the specimen in two with a cheese wire, and smoothing the surfaces on a glass plate, before re-assembly. The back of the specimen may need to be packed out slightly at this stage, to ensure that the soil is slightly proud of the edges of the box once the vertical load is re-established. The effective angle of friction produced by this method of specimen preparation is thought to be slightly less than that obtained from ring shear testing.

Ring shear testing of clay, both in the Imperial College / NGI apparatus and the Bromhead apparatus, tends to give values of residual angle of friction that are, according to Skempton (1985) 1° or 2° below the values back-calculated from field observations, despite the fact that the rate of shear is higher in the laboratory. For the London Clay, Bishop et al. (1971) found ϕ'_r to be 14° at low effective normal stress (< 30 kPa), reducing to less than 9° at effective normal stresses greater than 100 kPa.

Based on data from Petley (1966) and Lupini (1980) Skempton noted that a change in measured strength of 'rather less than 2.5% per log cycle' of deformation rate, and concluded that rate effects will be negligible within the usual rates of testing (0.002–0.01 mm/minute). Because there is no control over the gap in a Bromhead ring shear apparatus, it is necessary to run the test at a rate fast enough to limit loss of fines. For this reason, and because of the limited rate effect, it has proved convenient to test at a constant rate (about 0.04 mm/minute) carrying out an entire test consisting of 4 or 5 loading stages within a 24-hour period (see Clayton et al., 1995, p. 427). Total normal stresses are applied without stopping rotation, and drainage is deemed complete when the measured shear stress becomes constant. This is then held constant for 20 minutes before the next loading stage is applied.

The use of the correct effective vertical effective stress is essential in all residual shear strength testing, since in common with peak strength envelopes, residual strength envelopes show significant curvature at low effective stress levels. Skempton (1985) notes that for most clays the relationship between residual strength and normal effective stress is non-linear, and varies from clay to clay. For the clays he considers the residual effective angle of friction rose, on average, by some 17% as the normal effective stress dropped from 150 kPa to 25 kPa.

7.4 *Hollow cylinder testing*

Recent HCA testing of intact samples of natural soils at Imperial College has involved the use of the smaller devices (100 mm OD by 70 mm ID and 70 mm OD by 38 mm ID) on block and rotary core samples of Bothkennar clay (Albert et al, 2003), London Clay (Nishimura et al, 2006) and Pentre clay (Porovic, 1995). Special techniques are required in the preparation of the hollow cylindrical sample for testing. Nishimura et al. describe the procedure as follows: 'clay samples were firstly trimmed to the specified outer diameter and height with a wire saw in a rotary soil lathe. The solid cylindrical specimen thus formed was then placed in a metal mould and transferred to a metal working lathe. While the specimen was rotated at 190 rpm, a set of six drill bits was advanced from one end. The process involved gradually increasing the bit diameters until the desired inner diameter was achieved. For drier samples (including all the block samples and deeper rotary core samples), the friction between the inner clay surface and the larger-diameter drills tended to open natural discontinuities and disturb the specimen. For such samples, the final drilling stages were replaced by careful point-edge reaming to reach their final diameter.' In tests on reconstituted materials Grabe (2002) used a similar technique, but was able to make the central hole manually with a wood auger whilst continuing to hold the specimen in the soil lathe.

In the experiments the effects of principal stress direction, and in some cases the effects of the relative magnitude of the intermediate principal stress, were investigated. The HCA overcomes some of the shortcomings of the conventional simple shear devices and has been used in the past to examine behaviour in simple shear (Shibuya and Hight, 1987; Pradhan et al, 1988). Nishimura et al (2006) report the results of such tests on block samples of London Clay,

7.5 *Permeability testing*

Providing that the insertion of the testing device causes minimal disturbance to the surrounding soil, for example by using a self-boring permeameter, it is probably true that in-situ values of permeability are

best measured in in-situ tests, since these test a large volume of the soil. The advantages of laboratory permeability testing is that it can be used to determine permeability anisotropy, i.e. $r_k (=k_{vo}/k_{ho})$, and the change in permeability with void ratio, i.e. the permeability change index $C_k (= \Delta e / \Delta \log k)$.

Laboratory measurements of permeability are made in a range of apparatus, depending on the soil type and its permeability. The permeability of granular soils is usually measured in a constant head permeameter having a diameter of 75 mm or 114 mm and the test is run according to ASTM D2434 (Permeability of Granular Soils (Constant Head)) or BS1377:1990 Part 5, Section 5, which suggests that the cell diameter should exceed the maximum particle size of the soil being tested by a factor of 12.

The equipment used to measure the permeability of silts and clays in the laboratory include:

- The conventional oedometer, which can be used to measure k_{vo} and Ck_v on samples trimmed horizontally, and to measure k_{ho} and Ck_h on samples trimmed vertically
- The Rowe hydraulic consolidation cell which can be used to measure k_{vo} and Ck_v and k_{ho} and Ck_h on samples of 75 mm, 150 mm and 250 mm trimmed horizontally. For vertical permeability, porous discs are provided at the top and bottom of the specimen. For horizontal, or more correctly radial, permeability, a peripheral drain of porous plastic is fitted inside the oedometer ring and a porous plastic tube is installed in the centre of the specimen. In their tests on Bothkennar clay Little et al. (1992) report using a 11.1 mm OD plastic tube when testing 75 mm diameter specimens, and 25 mm OD plastic tube when testing 150 mm diameter samples.
- The radial flow cell (Leroueil et al. 1990) in which a central injection hole of 50 mm is cut into specimens of 180 mm diameter by 90 mm high and measurements made of radial permeability under different stress states.

Tests on clays may be run as constant head tests, falling head tests or constant flow tests using a flow pump.

The value of the coefficient of permeability measured in the laboratory is influenced by, inter alia:

- The size of the test specimen, in particular its representativity in terms of fabric features. Of particular importance in the measurement of horizontal permeability is the diameter of the specimen in relation to the persistence of any permeable seams.
- A lack of saturation in the test apparatus and connecting leads. The advantage of hydraulic oedometers, such as the Rowe cell, over the conventional oedometer is the ability to run permeability test using a back pressure to reduce the risk of air bubbles in the system and to ensure saturation of the specimen.
- The chemistry of the permeant and the permeated volume. The larger the ratio of permeated volume to the volume of pore water in the specimen, the more important the permeant. Obviously it is preferable to use a permeant having the same chemistry as that of the liquid that will pass through the soil in situ. When this is not possible it is preferable to use de-aired water to reduce the risk of air bubbles in the system.
- The temperature and therefore the viscosity of the permeant. Permeabilities are usually measured in the laboratory at a higher temperature than applies in situ and measured values need to be corrected.

The conclusions reached by Tavenas et al. (1983) in their excellent review of laboratory measurements of the permeability of natural soft clays still stand and are worth repeating in summary form:

- Vertical permeability can be measured reliably in constant head tests in the triaxial cell, measuring both inflow and outflow and ensuring that all system leakages are avoided, osmosis and diffusion are kept to a minimum, and pre-soaked membranes are used. It may be necessary to set a minimum value to the effective radial stress to avoid leakage between the soil and the membrane. The advantages of using the triaxial cell are the range of sizes available, the ability to test samples up to 200 mm, and the ability to apply anisotropic stresses and back pressures.
- Falling head tests are preferable when using the oedometer apparatus.
- Permeabilities derived from C_v values in oedometer and CRS consolidation tests are unreliable, tending to be too low in incremental load and controlled gradient tests, and too high in constant rate of strain tests.

7.6 Testing unsaturated soils

Test methods for unsaturated soils have been discussed by many authors, and reviewed by Fredlund and Rahardjo (1993) and will not be repeated in detail here. As far as the authors are aware, there are no currently accepted standards for the routine testing of unsaturated soils.

Techniques for triaxial testing are discussed in their state-of-the-art report by Hoyos et al. (2005), who consider the following test stages:

- Equalisation
- Isotropic compression
- Shearing.

They report that during initial equalisation the total stress, σ , and pore air pressure, u_a , are usually increased together, with the specimen undrained, and that the required pore water pressure, u_w , is then applied. This system will work at low degrees of saturation, where there is continuous pore air, but that at higher degrees of saturation when there is occluded air in the pore water the application of the total stress may lead to an initial volumetric reduction of the specimen, which presumably could lead to changes in subsequent behaviour, despite subsequent stabilisation.

Isotropic stress application can be carried out either in steps or continuously. Step loading has the advantage that it can be applied manually, and that steps can be held until the specimen stabilises, but it has disadvantage that excess water pressure can be induced by each step, leading to overestimates of soil compressibility. It is suggested that rates of isotropic stress application are checked by occasionally stopping and observing whether changes in specimen volume and water volume continue to occur.

Similarly, a suitable empirical method for determining a reasonable rate of shearing is by stopping the application of deviatoric load and observing whether significant changes in specimen volume and water volume continue. Hoyos et al. comment that a shearing stage taken to failure can typically take 1–2 weeks, and that suitable means of determining constant water (undrained) shearing rates have yet to be determined.

8 CONCLUSIONS

This paper has reviewed soil testing on the basis of the authors' experience, and has highlighted issues that may affect the success of laboratory soil testing. A very wide range of testing apparatus, instrumentation and test methods is currently available. The results that are obtained depend upon the combination of soil disturbance, apparatus, and instrumentation, and test method, and can vary very considerably as a result of some details in these. Apparatus and test method continue to develop, particularly for the testing of unsaturated soils. Geotechnical testing is becoming more complex, but at the same time the crucial knowledge base of experienced engineers and technicians involved in commercial testing is reducing.

If engineers are to have confidence in the results produced by laboratory testing then it is essential that results can be checked. As the few instances reported in this paper show, round-robin testing can make an important contribution to our understanding of repeatability. Tests results for standard materials also help those setting up testing facilities for the first time to check the validity of their work. There is a need to develop a central database of test materials, detailed techniques and results to support the constructive development of geotechnical soil testing.

ACKNOWLEDGEMENTS

The authors are grateful for the advice given by colleagues in the UK and around the world, and in particular Dr. Beatrice Baudet (UCL, London), Professor Adriano Bica (UFRGS, Brazil), Dr. Angus Best, Dr. Hannes Gräbe (Spoornet, South Africa), Prof. Gerhard Heymann (University of Pretoria), Professor Richard Jardine (Imperial College, London), Dr. Jeffrey Priest, Mr. Hardev Sidhu (SGC, UK), and Dr. Li-Ang Yang (University of Southampton).

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In-situ test calibrations for evaluating soil parameters

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ABSTRACT: The interpretation of in-situ geotechnical test data needs a unified approach so that soil parameters are evaluated in a consistent and complementary manner with laboratory results. A common thread in assessing in-situ tests is the focus on the geologic stress history, often expressed by the overconsolidation ratio (OCR). For clays, the OCR can be measured by consolidation tests on undisturbed samples, yet for sands is rather problematic to address. For 6 clays, a hybrid cavity expansion – critical state model is used to match responses measured CPT, CPTu, and DMT. Specifically, tip stress, sleeve friction, penetration porewater pressures, and flat dilatometer readings are fitted by parametric input of OCR, void ratio, friction angle, rigidity index, and compressibility parameters. The undrained shear strength (s_u) of clays is best handled via critical-state concepts. Discussions are included for pressuremeter, vane, and T-bar tests. For sands, select empirical methods derived from laboratory chamber testing on reconstituted clean quartz and siliceous sands are reviewed, specifically for effective friction angle ϕ' , OCR, and K_0 . In a novel look, a special set of undisturbed (frozen) sand samples from 15 locations in Japan, Canada, Italy, Norway, and China is used to check interrelationships for the following in-situ penetration tests: SPT, CPT, and V_s . Stiffness of all soils begins with the small-strain shear modulus ($G_0 = G_{\max} = \rho_T V_s^2$) that can be used together with strength (s_u or ϕ') to evaluate stiffness over a range of strains. Supplementary testing by PMT and/or DMT can provide intermediate stiffnesses for tuning of modulus reduction schemes, as well as independent assessments of K_0 and OCR.

1 INTRODUCTION

1.1 *Natural geomaterials*

Geomaterials encompass an extremely wide range of natural soils, rocks, and intermediate earthy substances, as well as artificial fills, mine tailings, and slurries (see Figure 1). Grain sizes of these materials vary tremendously from nano-particles in the very small angstroms domain and colloidal range (<0.5 microns), to clay particle sizes (<2 microns), silts (<75 microns) to coarse-grained soils (>0.075 mm) including sands to gravels, as well as much larger cobbles (150 to 300 mm) and boulders (>300 mm) to fractured rocks and massive intact rock formations (1 m to 1 km). These geomaterials are generally very old (thousands to millions of years old) with the youngest generally represented by Holocene age formations that have been placed only within the last several thousand years ($<10,000$ years).

Natural geomaterials have been formed by an assortment of geologic processes including water sediment (marine, lacustrine, estuarine, alluvial, deltaic, fluvial), ice (glacial), wind (aeolian), disintegration (residual), mass wasting (colluvial), chemical (carbonates, calcarenites), biological (organic), and other mechanisms (e.g., meteoric). The varied mineralogies (e.g., quartz, feldspar, mica, chlorite, illite, smectite, kaolinite, montmorillonite, hallosite, carbonates) can occur in unlimited combinations and permutations, thus forming infinite possible grain size distributions, as well as variances in plasticity, particle shape, size, roundness, packing, porosity, and fabric. These enumerable facets could be construed to defy any notion that characterization of these materials is at all possible.

After initial placement or forming, these materials are subjected to a wide range of environmental, geomorphological, and climatic changes including erosion, desiccation, ageing, wet-dry cycles, seasonal temperature fluctuations, and in some cases, special events such as glaciation, cyclic loading due to earthquakes, load fluctuations due to wave and tidal activities, and other activities. As a consequence, most soils are at least slightly to moderately overconsolidated to some degree. The significant fact that

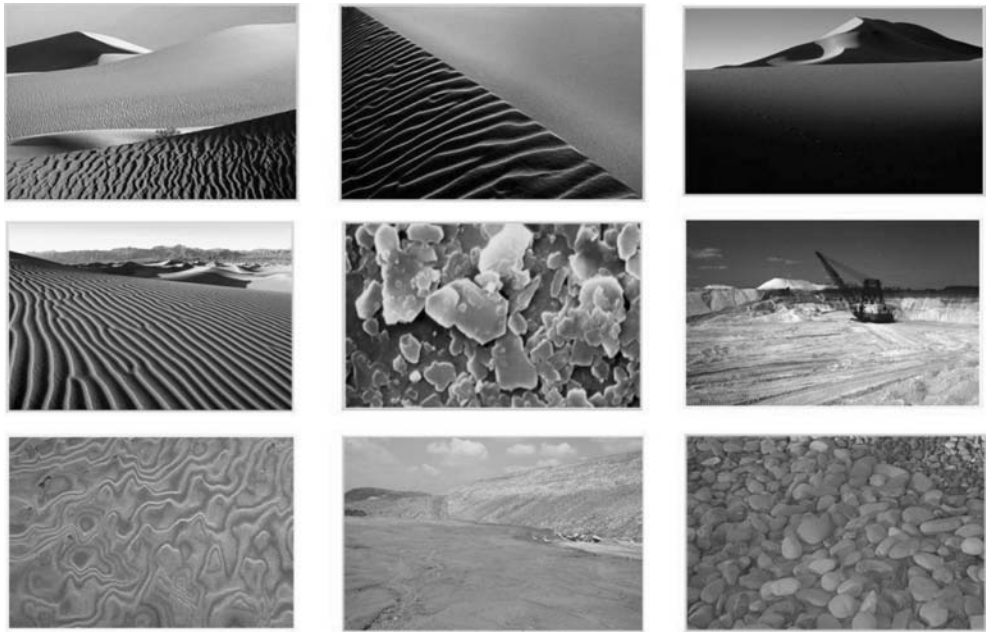


Figure 1. Illustrative examples of diversity in natural geomaterials found on planet Earth.

sea level has risen 30+ m in recent geologic times (thus associated groundwater tables), is sufficient alone in causing appreciable preconsolidation in soils (Locat, et al. 2003).

1.2 Site investigations

A proper characterization of natural geomaterials is paramount in all site investigations because the results will impact the solution with respect to safety, performance, and economy. For instance, if the site characterization program has collected poor quality undisturbed samples of clays & silts for laboratory consolidation and triaxial shear testing, the soil stiffness and strength will be underestimated, perhaps resulting in the requirement of driven pile foundations for the building structure whereas shallow footings could have adequately sufficed. In site investigations of clean sands that are most difficult to sample, the assumption of normally-consolidated conditions will inevitably lead to an underestimate in pile side friction or an overestimate in shallow footing settlements. Therefore, a knowledge of in-situ test methods and their interpretation is of great value to the practicing geotechnical engineer in order to place the best value in the applied solution in site development.

A number of different approaches are available for the assessment of ground conditions at a particular site. These might be grouped into three categories, in preferred order:

- A. Geophysics for general mapping of the relative variances across the site.
- B. In-situ tests for profiling vertical geostratigraphic changes and/or soil parameter evaluation.
- C. Drilling and sampling to obtain high quality and representative materials for the laboratory.

An integrated approach should be adopted for studying the ground whereby non-destructive geophysics are used first to guide the selection of expedient probe sounding locations, which in turn would aid in the choice & direction of the more expensive & laborious soil test borings to provide undisturbed samples for controlled laboratory testing by triaxial, simple shear, consolidation, and/or resonant column devices. A versatile and well-calibrated constitutive soil model would link these aspects together in a rational and consistent framework. A full discussion on the basic laboratory, field, & geophysical methods and their procedures is beyond the scope of this paper but may be found elsewhere (e.g., Mayne, Christopher, & DeJong, 2002).

A large number of in-situ devices and field tests are available to delineate the geostratigraphy and determine specific engineering parameters in the ground. As depicted by selected tests in Figure 2, these

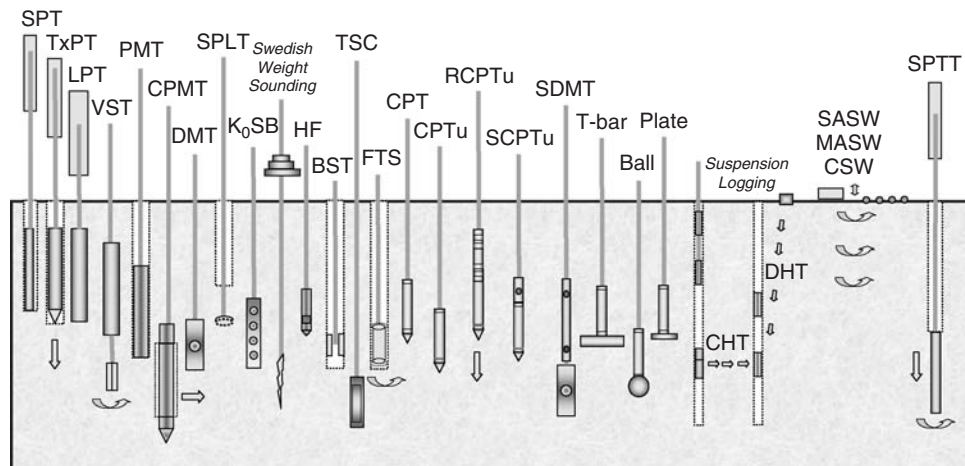


Figure 2. Selection of available in-situ geotechnical tests for determination of soil parameters.

are quite numerous and include: standard penetration, cone penetration, dilatometers, pressuremeters, vanes, flat and stepped blades, hydro-fracture, borehole shear, torsional probes, and many other innovative designs. Robertson (1986) provides a detailed overview on many of the standardized and specialized devices that are primarily penetration type and/or direct-insertion type devices for testing the ground. Woods (1978) and Campanella (1994) give a review of applicable geophysical tests for determining mechanical wave properties. Wroth (1984, 1988), Jamiolkowski et al. (1985), and Lunne et al. (1994) provide summary papers concerning the use & interpretation of the more common standardized tests and recent updates have been made by Yu (2004), Mayne (2005), and Schnaid (2005). An optimization of site-specific site investigation is achieved with seismic piezocone testing (Campanella et al. 1986) together with intermittent dissipation testing (SCPTu), since five independent readings are obtained with depth in the same sounding (Mayne & Campanella 2005): q_t , f_s , u_b , t_{50} , and V_s . A similar procedure can be provided by the seismic dilatometer test (Mlynarek et al. 2006) with A-reading dissipations (SDMTa) to obtain: p_0 , p_1 , p_2 , t_{flex} , and V_s .

1.3 Conventional approach for clays

In the conventional and classical methods of interpretation, the results of in-situ tests are commonly divided into two categories: (a) clays, whereby the undrained shear strength (s_u) is assessed; and (b) sands, whereby the relative density (D_R) and/or effective friction angle (ϕ') is evaluated. Yet, within the framework of critical state soil mechanics (CSSM), all soils in fact are frictional materials and their strength envelope can be best represented by their effective stress friction angle (ϕ'). For most soils, it can be taken that the effective cohesion intercept $c' \approx 0$ (unless true cementing or bonding is present). As these geologic materials are quite old and have been lying uninterrupted for long eons of time, the drained strength, in most cases, is quite characteristic of their "normal" behavior. With the introduction of man's activities involving construction on the ground, however, short term conditions can result in what might be termed "undrained" loading, corresponding to shearing under constant volume. This undrained condition is critical for normally- to lightly-overconsolidated soils of low permeability (i.e., soft clays & silts) under relatively fast (geologically speaking) rates of loading. Notably, the undrained conditions are merely transient, and given time, excess porewater pressures will eventually dissipate to equilibrium conditions (i.e., hydrostatic porewater pressures, u_0).

For clays, one difficulty with the conventional interpretation methods involving in-situ tests is that a reference benchmark value of s_u is required in the calibration and/or verification of direct probe tests, such as the standard penetration test (SPT), cone penetration test (CPT), piezocone (CPTu), flat dilatometer test (DMT), and other devices (e.g., T-bar, ball penetrometer). The dilemma is depicted in Figure 3. The appropriate mode of undrained shearing is not always known at the time of interpretation and might include triaxial compression (CAUC, CK_0UE , CIUC), plane strain compression (PSC), simple shear (SS), torsional shear (TS), or one of the extension modes (CAUE, CIUE, PSE, CK_0UE). Moreover,

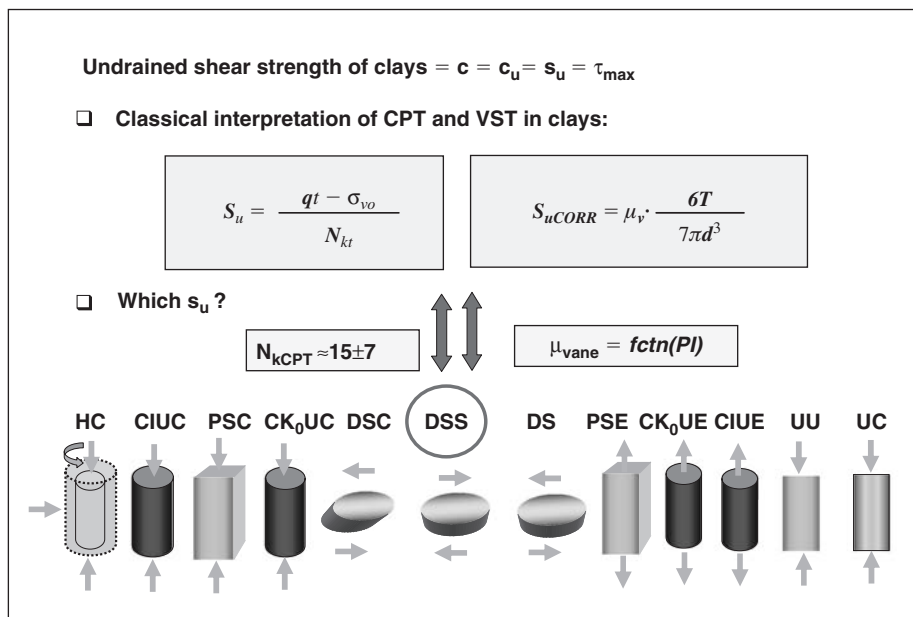


Figure 3. Dilemma in matching laboratory benchmark mode for undrained shear strength (s_u) with in-situ CPT and VST data. Note: Direct simple shear (DSS) is the likely reference for stability problems.

multi-modes or non-standard modes may actually apply. In consideration of these possibilities, it is not at all clear how stress state, anisotropy, and direction of loading affects the interpretation of s_u from each of the field tests.

Another problem lies in the inconsistency amongst interpretative frameworks for in-situ tests. Specifically for clays, the vane shear test (VST) is analyzed using limit equilibrium methods, whilst the pressuremeter test (PMT) is assessed within cylindrical cavity expansion theory, and yet the bearing factor term for interpretation of the cone penetration test (CPT) might come from either strain path method or finite elements solutions. For a given parameter (i.e. s_u), the different theories alone will provide inconsistencies in interpretations amongst the various in-situ tests, within the same soil formation.

Each of the in-situ test types is conducted at a different rate of strain, thus affecting compatibility in the comparison of different tests. For example, at a rate of penetration of $v = 20$ mm/s, the CPT pseudo-strain rate could be considered to be v/d , where the diameter of a standard 10 cm² penetrometer is $d = 35.7$ mm. This pseudo-strain rate is 56%/s, or about $2 \cdot 10^5$ %/hour. In comparison with a standard laboratory triaxial compression test performed at 1%/hour, this is over 5 orders of magnitude faster! For the aforementioned reasons of strength anisotropy, strain rate, boundary conditions, as well as other factors (e.g., progressive failure, drainage, fissuring), empirical factors have been included in the interpretation of undrained strengths from in-situ tests. Notably, the vane shear strength (s_{uv}) is often corrected using a factor (μ_v) obtained from the clay plasticity index (PI) to obtain the operational undrained shear strength for stability analysis: $s_{ucorr} = \mu_v \cdot s_{uv}$ (e.g., Chandler, 1988). Likely, this procedure provides a means of converting the vane data to a representative mode, such as direct simple shear (DSS). Yet, most commercial labs do not own nor operate DSS equipment. Consequently, practitioners will often compare the derived s_u values from the in-situ measurements with simpler tests that they can perform in-house, such as unconfined compression (UC), unconsolidated undrained (UU) triaxial, as well as very poor quality index values from the lowly pocket penetrometer and/or torvane devices. Occasionally, results from higher quality tests, such as consolidated triaxial compression tests (CIUC) will be available by the practicing engineer, but these provide values some 30% to 60% higher than the DSS mode. In terms of critical-state soil mechanics (CSSM), stress-induced anisotropy can be considered and shown to provide a reasonable hierarchy amongst the various lab testing modes in providing values of the normalized undrained shear strengths (s_u/σ'_{vo}). However, inherent anisotropy or fabric-induced features may not be well covered by simple constitutive soil models.

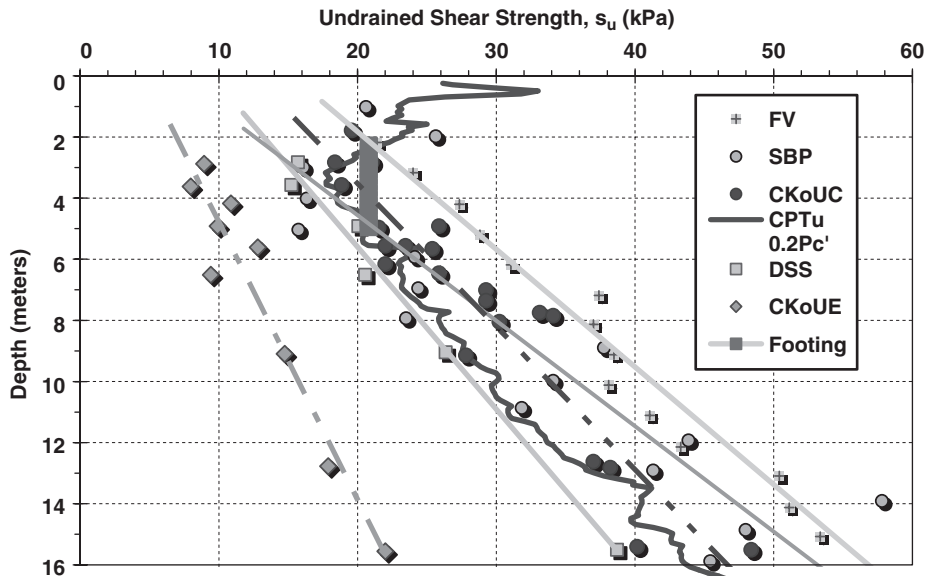


Figure 4. Different mode profiles of undrained strengths at Bothkennar clay site, UK.

Using data from the British national experimental test site located at Bothkennar, Scotland, Figure 4 shows the various derived undrained shear strengths from laboratory triaxial compression, simple shear, and extension tests on undisturbed samples (Hight et al. 2003) together in comparison with direct measured values from self-boring pressuremeter (Powell & Shields, 1995) and field vane tests (Nash, et al. 1992). Quite a variation is seen for all depths. Also indicated is the backfigured operational value of s_u from a full-scale footing load test ($B = 2.4$ m) conducted at the site using a limit plasticity solution with a bearing factor $N_c^* = 7$, as reported by Jardine, et al. (1995).

1.4 Conventional approaches for sands

For sands, the major difficulty lies in the inability to collect true undisturbed samples from the field in order to establish a clear laboratory calibration value of strength or stiffness whatsoever. Therefore, primary efforts have resorted to collecting disturbed bulk samples of sands from the field and reconstituting the sand specimens at the “same density” in the laboratory. Presumably, the in-place density has been accurately assessed and the lab results give stress-strain-strength behavior that can be linked to the field tests. The reconstituted sands can either be formed into: (a) small triaxial- or direct shear-size specimens that are related back to field penetrometer readings, or (b) large diameter calibration chambers, mostly of the flexible-wall type, that allow the full probing by the penetrometers or in-situ tests. Dry to saturated sands under normally-consolidated to induced overconsolidated states have been prepared in this manner under different boundary conditions (Ghionna & Jamiolkowski 1991). After all is known beforehand about the chamber deposit of sand (particle shape & angularity, median particle size, percent fines, density, stress history), a miniature to full-size probe is inserted for the test performed (Figure 5). Laboratory calibration chamber tests (CCT) have included SPT, CPT, DMT, PMT, CPMT, and mechanical wave geophysics for compression or P-wave and shear or S-wave velocity measurements.

It is well-known that the CCT data suffer from their limited boundary sizes represented by the ratio D/d , where D = diameter of large sand sample and d = diameter of probe. Thus, the results need be corrected to far-field values (e.g., Salgado, et al. 1997; Jamiolkowski et al. 2001). Moreover, different preparation methods for placement of “sand deposits” have been used in CCT, including: compaction, air pluviation, water pluviation, moist tamping, vibration, and slurring. For a given density of the sand, these in fact do not provide similar fabrics and thus the obtained probe results may differ significantly from field behavior (e.g., Hoeg et al. 2000; Jamiolkowski 2001). Finally, the supposed link in the relationships derived from the CCT correlations derives from the assumption that the in-situ tests can provide a direct measure of the relative density (D_R). In US practice, the notion of D_R only applies to cases of clean

Calibration Chamber Testing

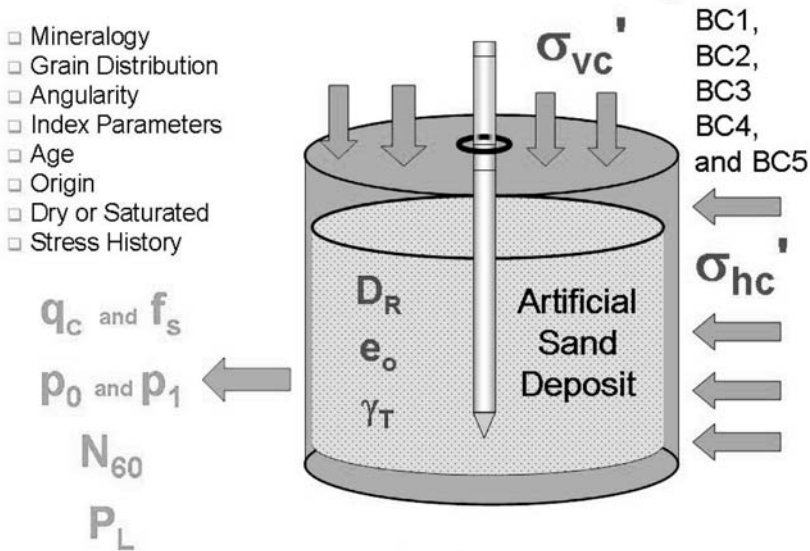


Figure 5. Calibration chamber testing setup for in-situ testing on reconstituted sands.

sands with percent fines $PF \leq 15\%$. In truth, the use of in-situ penetration measurements can provide only a rough index on the in-place packing arrangement such as void ratio, porosity, or relative density. Thus, other devices such as gamma logging, downhole nuclear gages (i.e., radio-isotope penetrometers), or time domain reflectometry (TDR) in direct push probes would serve better for assessing this initial state condition.

1.5 Fitting & calibration approaches

Herein, a couple of non-conventional approaches are presented for evaluating in-situ test results in clays and sands. The emphasis is initially on the CPT, CPTu, and DMT for clays, with dissipation response also considered; and SPT, CPT, and V_s testing in sands. The methods are not rigorous but utilized to show an important geotechnical need regarding a full set of complementary calibrations of in-situ tests within a soil medium.

For the clays, a known profile of stress history (i.e., OCR) is used to drive the fitting of in-situ penetration data in terms of simple closed-form expressions derived from hybrid cavity expansion – critical state models. Input parameters include the initial state (void ratio, e_o), effective friction angle (ϕ'), plastic volumetric strain potential ($\Lambda = 1 - C_s/C_c$), and rigidity index ($I_R = G/s_u$). Reasonably successful forward profiles of cone tip stress (q_t), penetration porewater pressures (u_1 and/or u_2), sleeve friction (f_s), dilatometer contact (p_0) and expansion pressures (p_1) are shown for three soft clay sites and two overconsolidated clays. Research needs include a generalized methodology for all types of in-situ tests that is internally consistent and then calibrated with well-researched test sites, as those reported in the 2003 and 2006 Singapore Workshops (Tan, et al. 2003).

For sands, a two phase study is given: (1) examination of selected existing relationships for stress history (K_0 and OCR) and friction angle (ϕ') at three documented sand sites; and (2) a specially-compiled triaxial database created from expensive undisturbed (primarily frozen) sand samples where in-situ standard penetration, piezocone penetration, and shear wave measurements were collected. These latter series document the measured void ratio (e_o), relative density (D_R), triaxial friction angle (ϕ'), and stiffness from stress-strain curves. The parameters can then be compared with their corresponding in-situ measurements of N-value, tip stress q_t , and downhole V_s values in terms of existing correlative trends, theoretical relationships, and/or to allow future calibration with soil constitutive models.

2 IN-SITU TESTS IN CLAYS

2.1 Overview

In this approach, simple analytical models will be matched to the measured in-situ field data based on its stress history profile. Soil input parameters are parametrically varied to provide the optimal fitting to all available tests. At this time, the author has experimented with a simple hybrid model based primarily on spherical cavity expansion and critical-state soil mechanics (CSSM) that can be applied to cone penetration, porewater pressures, and flat dilatometer tests in clays. Brief discussions are given subsequently on the topics of stress history, cavity expansion, and CSSM.

2.2 Stress history

The benchmark for the preconsolidation stress is that caused by mechanical processes, specifically the removal of overburden, as occurs by erosion, glaciation, and/or excavation (Chen & Mayne 1994; Locat et al. 2003). Yet, effects of overconsolidation that are caused by a rise in the groundwater table, desiccation, wet-dry cycles, freeze-thaw cycles, and/or quasi-preconsolidation due to secondary consolidation, creep, and/or ageing might also be ascertained from these approaches, at least on an approximate basis.

For clays, the preconsolidation stress ($\sigma'_p = \sigma'_{vmax} = P'_e$) can be uniquely determined as the yield point on conventional one-dimensional consolidation plots of void ratio vs. log effective stress (i.e., e-log σ'_v data). The normalized form is termed the overconsolidation ratio, $OCR = (\sigma'_p/\sigma'_{vo})$. Because of sample disturbance issues, the clear demarcation of σ'_p may be unclear or muddled, thus graphical “correction” schemes have been devised to better delineate its value (e.g., Grozic and Lunne 2004).

The OCR governs both the normalized undrained shear strength (s_u/σ'_{vo}) and the lateral geostatic stress coefficient, $K_0 = (\sigma'_{ho}/\sigma'_{vo})$. The OCR has also been shown especially influential on laboratory-derived values of small-strain shear modulus ($G_0 = G_{max}$), Skempton's porewater parameter (A_f), and intermediate stiffness values of modulus and rigidity (E_u/s_u and E'/p'_0). For natural sands, the effective preconsolidation occurs similarly by the same basic mechanisms. In calibration chamber tests involving sands, the prestressing is induced artificially as part of creating the “deposit”, thus the OCR is known. In natural sands, however, it has been difficult to ascertain the OCR because of sampling problems and the very flat e-log σ'_v response of sands in one-dimensional compression tests.

In truth, the vertical preconsolidation stress is just one point of an infinite locus of memory on the three-dimensional yield surface (Leroueil & Hight, 2003). As such, it is possible to define additional points of yielding along specialized stress paths in the triaxial apparatus and define this complete yield surface. This yield surface is rotated and centered about the K_{0NC} compression line in MIT type q-p space and governed by the effective frictional characteristics of the soil (e.g., Diaz-Rodriguez, Leroueil, & Aleman, 1992). Thus, in sands, it is conceptually feasible to use the same penetration data (N_{60} , q_t , p_0) to define the centering of the yield surface, as well as the friction angle of the material, since the two are related. For clays, the in-situ data (VST, CPT, CPTu, SPT, DMT) relate to the effective preconsolidation stress, thus also to the yield surface. However, since these are relatively fast tests under undrained conditions ($\Delta V/V_0 = 0$), then additional data related to the generated excess porewater pressures are needed to define the effective frictional envelope (e.g., Senneset et al. 1989).

2.3 Cavity expansion theory

In cavity expansion, three possible geometric cases can arise during integration of the governing equations: (a) linearly, to create a wall; (b) radially, to form a cylinder; and (c) spherically, resulting in a ball. The use of cylindrical cavity expansion (CCE) theory for the reduction of pressuremeter test (PMT) data in clays has been well-appreciated for over 50 years (Gambin, et al. 2005). Assuming undrained conditions, four parameters can be individually assessed for each PMT including: lift-off pressure ($P_0 = \sigma_{ho}$), elastic shear modulus (G), undrained shear strength (s_u), and the limit pressure (P_L), such that:

$$P_L = \sigma_{ho} + s_u [\ln(I_R) + 1] \quad (1)$$

where $I_R = G/s_u$ = undrained rigidity index. Thus, all four parameters are interrelated via equation (1). Figure 6 shows a comparison of the graphically-interpreted limit pressure versus that calculated using (1) with data from 34 clays tested by self-boring pressuremeters (SBP), thus indicating the general validity of CCE.

Details on the PMT interpretative procedures are given elsewhere (e.g., Wroth, 1984; Briaud, 1995; Ballivy 1995; Fahey & Carter, 1993; Clark & Gambin, 1998). Alternate approaches for cavity expansion

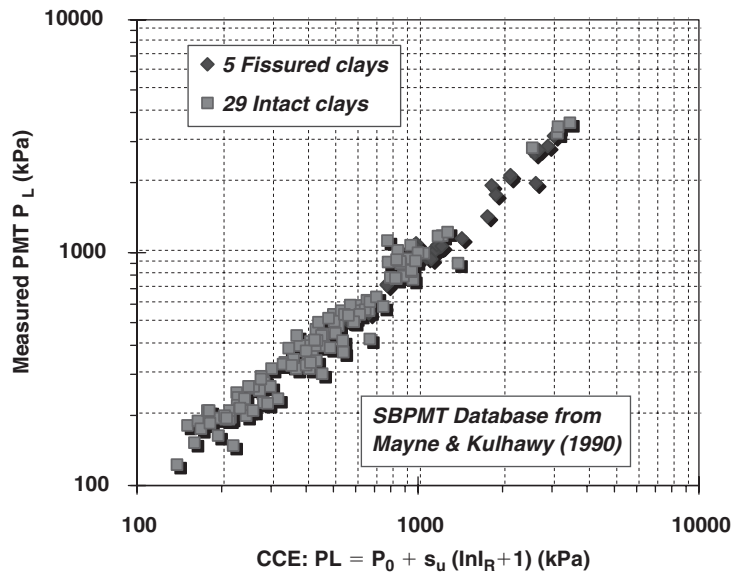


Figure 6. Measured vs. predicted limit pressure using SBPMT database in clays.

interpretation include the derivation of a continuous stress-strain curve from the measured pressure vs. volume change field data (e.g., Gambin et al. 2005).

The cavity expansion approach has been also extended to drained loading conditions by consideration of volumetric strain changes (Vesic, 1972, 1977), or alternatively using dilatancy angle (Carter, et al. 1986). Additional work has been undertaken to evaluate the unloading cycles during full-displacement type pressuremeters, such as the cone pressuremeter test (CPMT), as detailed by Houlsby & Withers (1988) and Yu (2004).

A parameter of interest to all cavity expansion problems is the rigidity index, defined as the ratio of shear modulus to shear strength, $I_R = G/\tau_{\max}$. The appropriate value of I_R is rather elusive since the shear modulus changes dramatically from an initial value ($G_0 = G_{\max}$) to a secant value at peak strength, $G_{\min} = \tau_{\max}/\gamma_f$, where γ_f = strain at peak. The tangent G is zero at peak strength. Notably, in SBPMT, the value of G could be reported from the first-time loading (i.e., backbone curve), or else from an applied unload-reload cycle (G_{ur}) that is considerably stiffer than the former. For penetration tests, the value is likely closer to $G_{\min}$. As illustrated by Figure 7, the rigidity index can also be considered as the reciprocal of the failure strain, or $I_R = 1/\gamma_f$.

2.4 Critical-state soil mechanics

Critical state soil mechanics (CSSM) is a valuable effective stress framework to interrelate concepts of frictional strength and consolidation (e.g., Schofield & Wroth, 1968). By specifying the initial state, CSSM can easily explain the differences between normally-consolidated (NC) and overconsolidated (OC) behavior, contractive vs. dilative response, undrained vs. drained strengths, positive vs. negative porewater pressure generation, as well as other behavioral facets such as partly drained to semi-undrained cases, cyclic behavior, creep, and strain rate (Leroueil & Hight 2003).

In the most simple version involving saturated soils, only three soil properties are considered (ϕ' , C_c , C_s) in addition to the initial state (e_0 , σ'_{vo} , and $\text{OCR} = \sigma'_p/\sigma'_{vo}$). Essentially, CSSM is the link between the well-known compression curves from one-dimensional consolidation tests (termed $e\text{-}\log\sigma'_v$ space) and the Mohr's circles from triaxial or direct shear tests (represented by shear stress vs effective normal stress plots, or $\tau\text{-}\sigma'_v$ space). These two spaces are depicted in Figure 8, along with a third diagram ($e\text{-}\sigma'_v$ space) that really offers no new information, just a way to follow the compression results in arithmetic scale for σ'_v and tie to the $\tau\text{-}\sigma'_v$ space.

In its essence, the premise for CSSM is that all soil, regardless of starting point or drainage conditions, strives towards and eventually ends up on the critical state line (CSL). In the $\tau\text{-}\sigma'_v$ space, this line corresponds to the well-known strength envelope given by the Mohr-Coulomb criterion for $c' = 0$. Here,

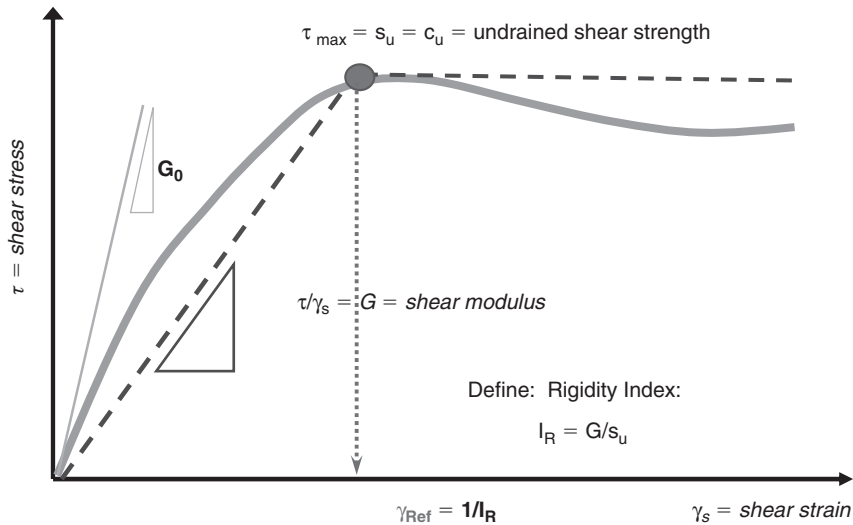


Figure 7. Shear stress vs. shear strain for soils and definitions of $\tau_{\max}$, G , γ_f , and I_R .

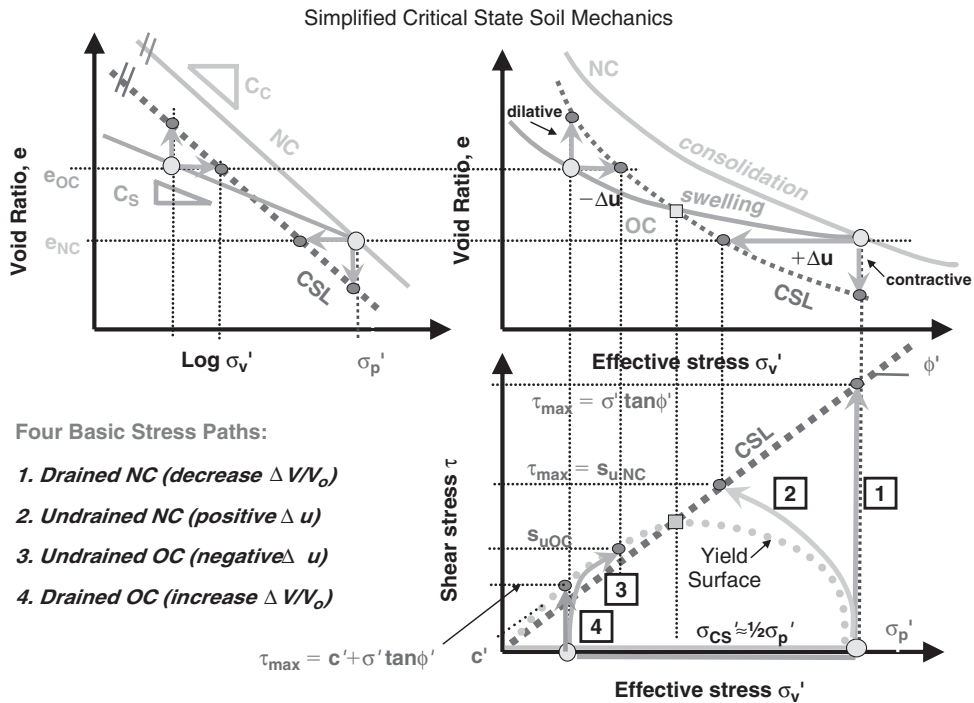


Figure 8. Simplified critical state soil mechanics, showing four common stress paths.

shear strength is represented by the maximum shear stress ($\tau_{\max}$) and is given by: $\tau = c' + \sigma'_v \tan \phi'$. In the e - $\log \sigma'_v$ space, the CSL represents a line parallel to the virgin compression line (VCL with slope C_c) for NC soils, yet offset to the left at stresses approximately half those of the VCL. All NC soils start on the VCL, while all OC soils begin from a preconsolidated condition along the recompression or swelling line (given by slope C_s). Regardless, all shearing results in peak stresses falling on the CSL. Figure 8 shows a summary of four basic stress paths corresponding to drained loading (no excess porewater pressures, or $\Delta u = 0$) and undrained loading (constant volume, or $\Delta V/V_o = 0$) for both NC and OC initial states.

Surprisingly, geotechnical practice has still not adopted the framework of critical-state soil mechanics (CSSM), yet the supporting research and clear evidence have been available for over a half-century (e.g., Hvorslev, 1960). A review of geotechnical textbooks used in the USA, for example, reveals that the most popular and best-selling books do not discuss nor even mention CSSM, the exceptions being those by Budhu (2000) and Lancellotta (1995). Of course, there are several excellent books specific to CSSM (Schofield & Wroth, 1968; Wood, 1990), yet omitted in geotechnical education in USA, China, and many other countries.

With a lack of background in CSSM, many practitioners still cling to the incorrect notion of running strength tests on three specimens to produce a set of total stress “c and ϕ ” parameters. For clays, the idea of “ $\phi = 0$ ” analysis still prevails in practice, yet all soils (clays, silts, sands, gravels) are frictional materials. The observation of “ $\phi = 0$ ” is really an illusion when porewater pressures are not monitored or not known, thus missing the CSL influence. As a consequence, the term “cohesion” is often used ambiguously, in some instances referring to the *undrained shear strength* ($c = c_u = s_u$), yet in other circumstances, intending to mean the *effective cohesion intercept* (c'). The former is obtained as the peak shear stress in a stress path of constant volume. The latter is obtained by force-fitting a straight line ($y = mx + b$) to represent the Mohr-Coulomb strength criterion ($\tau = \sigma' \tan \phi' + c'$) from laboratory strength data.

As noted previously, the strength envelope is actually more complex and best described by a frictional envelope (ϕ') having a superimposed three-dimensional curved yield surface that is governed by the preconsolidation stress, σ'_p (Leroueil & Hight, 2003). The shape, size, features, and movement of the yield surface distinguishes one constitutive model from another (e.g., Lade 2005), yet this facet is not necessary in order to convey the overall simplicity and elegance of CSSM (e.g., Lancellotta, 1995). In Figure 8, the basic concept of a yield surface is shown in the $\tau - \sigma'_v$ space. It is the intersection of this yield surface with the frictional envelope which dictates the offset distance of the CSL to the left of the VCL.

2.5 Undrained shear strength

The undrained shear strength is best represented in normalized form (s_u/σ'_{vo}) which is mode-dependent and reliant on initial stress state, strain rate, direction of loading, degree of fissuring, and other factors (Kulhawy & Mayne, 1990). The ratio s_u/σ'_{vo} can be evaluated using constitutive laws based on critical state soil mechanics (e.g., Wroth, 1984; Ohta et al. 1985; Whittle & Kavvas, 1994), or alternatively using empirical approaches such as SHANSEP (Ladd, 1991). For general use, the author subscribes to a three-tiered hierarchy, based on the availability of site-specific data (Mayne, 2003). Since direct simple shear (DSS) provides an overall representative mode for stability and bearing capacity analyses, the preferred approach is via CSSM (Wroth, 1984), as illustrated by Figure 9 and expressed by:

$$\left(\frac{s_u}{\sigma'_{vo}} \right)_{DSS} = \frac{\sin \phi'}{2} \cdot OCR^\Lambda \quad (2a)$$

where ϕ' = effective friction angle, $\Lambda = 1 - C_s/C_c$ is the plastic volumetric strain ratio, and C_s and C_c are the swelling and virgin compression indices, respectively. For intact natural clays, the full range of observed frictional characteristics vary from $18^\circ < \phi' < 43^\circ$ (Diaz-Rodriguez, Leroueil, & Aleman, 1992). Clays of low to medium levels of sensitivity exhibit values between $0.7 < \Lambda < 0.8$, whereas structured and sensitive soils show $0.9 < \Lambda < 1$.

If the intrinsic property values are not known, a default form is taken as (Ladd, 1991):

$$\left(\frac{s_u}{\sigma'_{vo}} \right)_{DSS} = 0.22 \cdot OCR^{0.80} \quad (2b)$$

which is empirically-based on four decades of experimental lab testing by MIT on various clays worldwide. Interestingly, (2b) is also a subset of (2a) for the case where $\phi' = 27^\circ$ and $\Lambda = 0.80$. Finally, if the material is lightly-overconsolidated with $OCR < 2$, the third tier estimate is simply (Trak, et al. 1980; Jamiolkowski, et al. 1985):

$$s_{uDSS} \approx 0.2 \sigma'_p \quad (2c)$$

which is based in good part on corrected vane shear data from backcalculated failures of embankments, excavations, and footings on soft clays.

If more specific modes are needed, these can be assessed using constitutive relationships (e.g., Wroth, 1984; Ohta, et al. 1985; Kulhawy & Mayne 1990), or empirical trends based on plasticity index

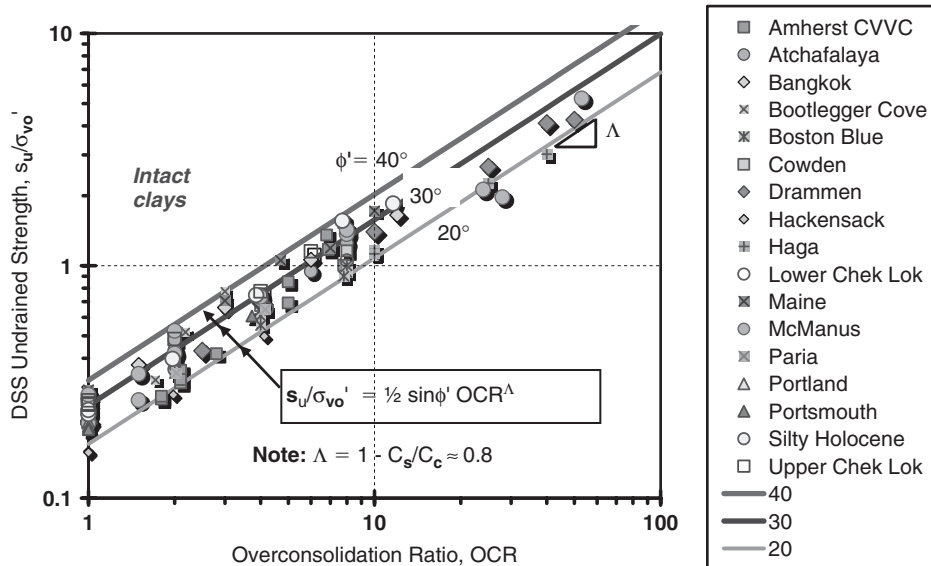


Figure 9. Experimental and CSSM variation of undrained DSS shear strengths with OCR.

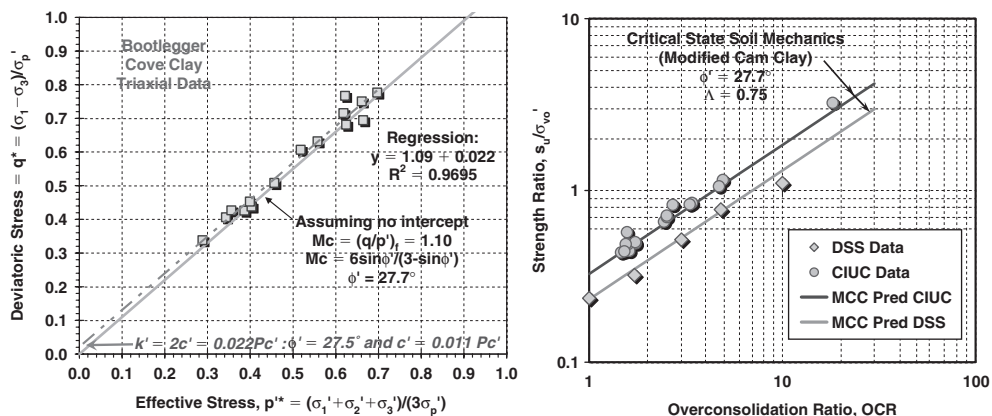


Figure 10. Measured and predicted s_u/σ'_{vo} vs. OCR for Bootlegger Cove Clay, Anchorage.

(e.g. Jamiolkowski et al. 1985; Ladd, 1991). For example, the triaxial compression mode (CIUC) that can be accommodated by most commercial laboratories is given by:

$$s_u / \sigma'_{vo \text{ CIUC}} = \left(\frac{M}{2} \right) \left(\frac{OCR}{2} \right)^\Lambda \quad (3)$$

where $M = 6\sin\phi'/(3 - \sin\phi')$ represents the frictional parameter in Cambridge $q - p'$ space. Thus, CSSM offers a nice framework to organize laboratory strength data in terms of the stress history of the clay, as well as a means of predicting changes in strength should onsite construction require the addition of new fill loading, embankments, and/or excavation. For illustrative purposes, Figure 10 shows a set of 15 CIUC triaxial tests and 5 DSS tests on Bootlegger Cove Clay from Anchorage, Alaska in terms of normalized undrained shear strengths vs. overconsolidation ratio (Mayne & Pearce, 2005). The data are well-represented by CSSM equations (2a) and (3) using the triaxial-determined friction angle $\phi' = 27.7^\circ$ and compression parameters from standard consolidation tests ($C_c = 0.24$; $C_s = 0.06$; $\Lambda = 1 - C_s/C_c = 0.75$).

Note that for fissured clays (generally associated with high OCRs), the values from (2a), (2b), and (3) should be reduced by as much as one-half because of the macrofabric of existing fractures and cracking. For soils that are overconsolidated by mechanical unloading, a limiting OCR is reached in extension when the lateral stress coefficient (K_0) reaches the passive value (K_p). For simple virgin loading-unloading of “normal soils” that are not highly cemented nor structured, the K_0 value increases with OCR according to:

$$K_0 = (1 - \sin \phi') \text{OCR}^{\sin \phi'} \quad (4)$$

If the passive condition is represented by the general condition, then K_0 cannot exceed:

$$K_p = \frac{1 + \sin \phi'}{1 - \sin \phi'} + \frac{2c'}{\sigma_{vo}'} \sqrt{\frac{1 + \sin \phi'}{1 - \sin \phi'}} \quad (5)$$

Generally, the value of effective cohesion intercept $c' \approx 0$, although a small finite value can be sometimes justified as a projection from the portion of the yield surface extending above the frictional envelope. In these cases, $c' \approx 0.02\sigma_p'$ may be applicable (Mayne & Stewart, 1988; Mesri & Abdel-Ghaffar, 1993).

2.6 Hybrid SCE-CSSM model for cone penetration

Cone penetration in clays can be modeled using a number of different theories (Yu & Mitchell, 1998), including limit plasticity (e.g., Konrad & Law, 1987), cavity expansion (Keaveny & Mitchell, 1986), and strain path methods (Whittle & Aubeny, 1993), as well as by finite elements (Houlsby & Teh 1988) and dislocation theory (Elsworth, 1998). Using a spherical cavity expansion solution (Vesic, 1977) with a CSSM representation of undrained loading for triaxial compression mode, Mayne (1991, 1993) showed that the cone tip stress could be expressed in closed-form by:

$$q_t = \sigma_{vo}' + [(4/3)(\ln I_R + 1) + \pi/2 + 1] \cdot (M/2)(\text{OCR}/2)^\Lambda \sigma_{vo}' \quad (6)$$

Similarly, the penetration porewater pressure at the cone shoulder ($u_b = u_2$) can also be expressed in terms of these parameters:

$$u_2 = u_0 + (4/3)(\ln I_R) \cdot (M/2)(\text{OCR}/2)^\Lambda \sigma_{vo}' + [1 - (\text{OCR}/2)^\Lambda] \cdot \sigma_{vo}' \quad (7)$$

Porewater pressures measured by type 1 cones (designated u_1) depend upon the actual position of the filter element (apex, below or above midface), as shown by Chen & Mayne (1994). An additional component representing compression beneath the tip can be simulated by an elastic stress path, such that for a typical midface element (Mayne, Burns, & Chen 2002):

$$\Delta u_1 = \Delta u_2 + (1/s^*) \cdot M \cdot (\text{OCR}/2)^\Lambda \cdot \sigma_{vo}' \quad (8)$$

where s^* is the slope of the total stress path in Cambridge q - p' space, averaging around $\Delta q/\Delta p' = 3/4$ for midface elements. A large database of 45 intact clays tested by paired type 1 and type 2 piezocones, or else by special multi-element penetrometers, shows good agreement for (5) in Figure 11 using representative values of $M = 0.92$, $\Lambda = 0.80$, and $s^* = 0.75$. Data from five fissured clays are also shown for comparative purposes. The fissured clays are of particular interest in that positive recordings are observed at the tip/face (u_1), whereas readings at the shoulder (u_2) are negative (Mayne, et al. 1990). Nevertheless, (8) is seen to provide a reasonable transform from u_2 to u_1 for these fissured clays too.

The sleeve friction in clays can be taken similar to a “Beta” method for calculating side friction of piles (e.g., Finno 1989). In this notion, the CPT sleeve resistance can be expressed by:

$$f_s = K_0 \tan \delta' \sigma_{vo}' \quad (9)$$

where δ' is the interface friction between the steel penetrometer and the soil. Based on recent interface roughness research, a representative value can be taken as $\tan \delta'/\tan \phi' = 0.4$. Using this with (4) can provide a first-order evaluation of CPT f_s in clays.

To illustrate the forward use of stress history to drive the readings of cone tip, porewater pressures, and sleeve friction, Figure 12 shows the matched profiles obtained from the fitting of CPTu data for Sarapu clay in Rio de Janeiro (data from Almeida & Marques, 2003) with parametric values: initial void ratio $e_0 = 3.0$, operational prestress $= \Delta \sigma_p' = 25 \text{ kPa}$, $\phi' = 29^\circ$, $\Lambda = 0.7$, and operational rigidity index $I_R = 50$. The groundwater lies 0.2 m deep and the void ratio generates an average (light) unit weight for calculation of the total and effective overburden stresses. The overconsolidation has been caused by

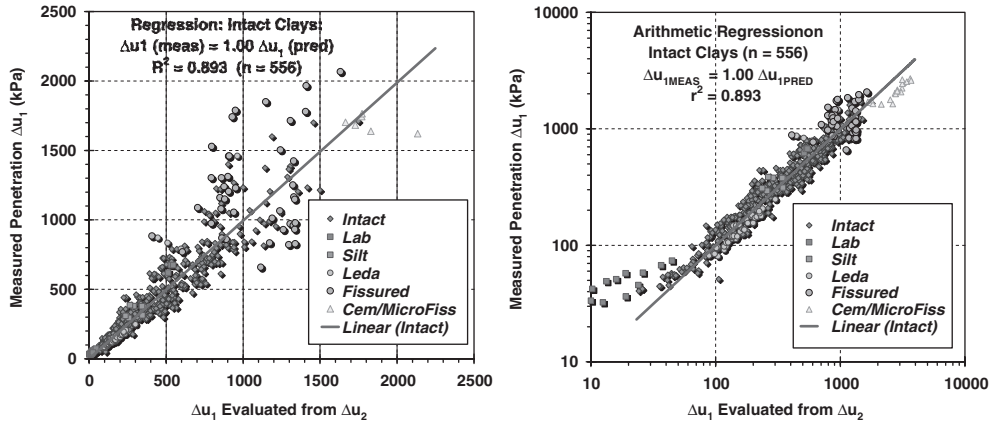


Figure 11. Measured midface u_1 porewater pressures from shoulder u_2 readings in (a) arithmetic and (b) log-log plots from 45 intact clays (data from 5 fissured clays also shown).

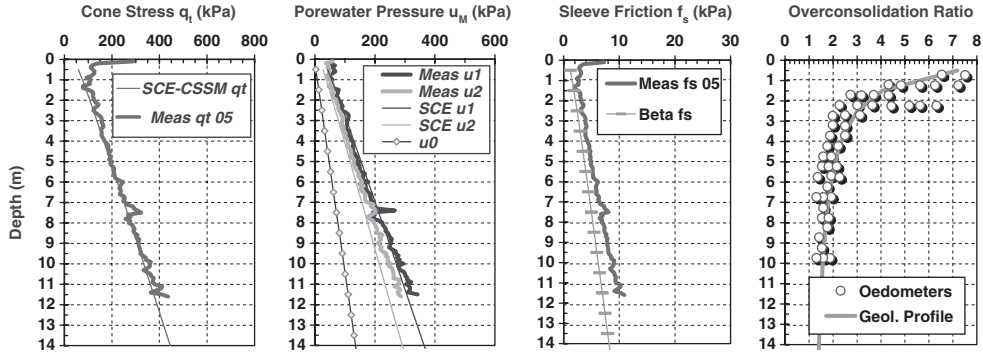


Figure 12. Generated and measured CPTu profiles in Sarapu soft clay, Brazil. (Data from Almeida & Marques, 2003).

ageing (Almeida & Marques, 2003), yet it suffices to use an equivalent calculated $OCR = [\Delta\sigma'_p/\sigma'_{vo} + 1]$. Then, equations (6) through (9) are adjusted to match the measured CPT readings with depth.

2.7 Hybrid SCE-CSSM model for dilatometer

Although the flat dilatometer geometry does not directly lend itself to application by cavity expansion theory, the initial generated porewater pressure isochones are reasonably symmetric about the blade at early stages of penetration (Huang, et al. 1991). Prior studies have shown considerable similarity between the initial lift-off or contact pressure (p_0) measured by the flat dilatometer test (DMT) and the measured porewater pressures during cone penetration (Mayne & Bachus, 1989; Mayne 2006a). Also, similarities exist between the DMT p_0 and limit pressure (P_L) from companion series of pressuremeter tests (PMT) in the same clays (Clarke & Wroth, 1988). Using the latter as a guide, let us adopt (1) as applicable to the DMT using p_0 to represent the limiting pressure:

$$p_0 = \sigma_{ho} + s_u [\ln(I_R) + 1] \quad (10)$$

We can assume the same value of rigidity index applies to the DMT as to the CPT, as both invoke similarly large levels of straining during installation. The total horizontal stress can be calculated as $\sigma_{ho} = \sigma'_{ho} + u_o$, using the fact that $\sigma'_{ho} = K_0 \sigma'_{vo}$.

Table 1. List of clay sites with force-fitted CPTu and DMT calibrations and derived parameters.

Site name	Location	z_w (m)	Void ratio e_0	$\Delta\sigma'_p$ (kPa)	OCR State	ϕ'	Λ	Rigidity I_R
Baton Rouge	Louisiana	5	0.92	850	4 to 15	28.5°	0.70	85
Bothkennar	Scotland	1	1.50	40	1 to 2	39.0°	0.90	85
Cooper Marl	S. Carolina	1	1.35	480	4 to 8	44.0°	0.90	750
Ford Center	Illinois	1	0.80	55	1 to 2	28.0°	0.80	300
Onsøy	Norway	0.5	1.75	30	1.2 to 3	32.0°	0.80	75
Sarapuí	Brazil	0.2	3.00	25	1 to 3	29.0°	0.70	50

Notes: z_w = depth to groundwater table; $\Delta\sigma'_p$ = prestress (also termed OCD by Locat, et al. 2003); OCR = overconsolidation ratio; ϕ' = effective stress friction angle; $\Lambda = 1 - C_s/C_c$ = plastic volumetric strain potential; C_s = swelling index; C_c = virgin compression index; and $I_R = G/s_u$ = operational rigidity index.

For the expansion pressure measured by the DMT, a similar stress path concept (as used previously for the CPT u_1 reading) can be assumed whereby:

$$p_1 = p_0 + M(OCR/2)^\Lambda \sigma_{vo}' \quad (11)$$

While (11) is far from rigorous, it does in fact provide reasonable values for the DMT material index, $I_D = (p_1 - p_0)/(p_0 - u_0)$ that is useful in classifying soil types and consistency.

2.8 Case study applications in clays

The simple representations for stress history ($\Delta\sigma'_p$, OCR) are used to generate full profiles of CPT q_t , f_s , u_1 , and u_2 , as well as DMT pressures p_0 and p_1 for three soft clays and two stiff overconsolidated soils. The soft clays include the UK national test site at Bothkennar (Hight, et al. 2003), the soft clay at Onsøy used by NGI (Lunne, et al. 2003), and a site very near the national geotechnical test site at Northwestern University (Finno, et al. 2000). For all these sites, a representative average void ratio and friction angle were known from lab tests on undisturbed samples taken from the site, as well as profiles of σ'_p from oedometer and/or consolidation tests. To check on the validity to stiffer clays, the method was also applied to two additional soils: (a) a sandy calcareous clay from Charleston, SC, known as the Cooper Marl (Camp, et al. 2002); and (b) stiff desiccated clay from Baton Rouge, Louisiana (Chen & Mayne, 1994). Table 1 summarizes the necessary input parameters that drive the complete analysis for each of the sites. Forward fitting of the SCE-CSSM algorithms for the in-situ tests in these six clays are presented in Figures 12 through 18. Brief descriptions of the clay sites, reference sources, and the calibrated fittings to in-situ test data are given below for each of the sites.

The *Bothkennar* clay site is located in Scotland and served as a major national test site for the British geotechnical community (Hight, et al. 2003). The entire June 1992 issue of *Geotechnique* is devoted to papers on the geology, geotechnical aspects, and lab and field testing in the soft Bothkennar clay (e.g. Nash, et al. 1992). The subsurface conditions consist of Holocene estuarine clays of 20-m thickness or more that have become lightly preconsolidated due to slight erosion ($\Delta\sigma'_p = 15$ kPa) and groundwater fluctuations ($1 < z_w < 3.5$ m), plus additional structuration, including ageing since deposition some 8000 to 3000 years before present. Index testing shows plasticity characteristics changing with depth: $50 < w_n < 75\%$, $25 < w_p < 35\%$; and $55 < w_L < 85\%$. The Bothkennar clay has a rather high effective stress friction angle at peak of between 36° and 45°. Using representative values of $\phi' = 39^\circ$ and $I_R = 85$, Figure 13 shows the forward fitting of the SCE-CSSM expressions for cone, piezocone, and flat dilatometer by adopting an equivalent prestress of $\Delta\sigma'_p = 40$ kPa. Here, additional types of piezocone data from u_1 (midface) and u_2 (shoulder) elements were also available (Powell, et al. 1988; Jacobs & Coutts 1992).

The soft *Onsøy* clay in Norway has served as an experimental test site for NGI for almost 40 years (Lacasse & Lunne, 1982; Lunne, et al. 2003). The site consists of approximately 40 m of uniform marine clay of Holocene age. Primary mechanisms for apparent overconsolidation include groundwater fluctuations and ageing. Index parameters for the Onsøy clay include: $50 < w_n < 70\%$; $w_p \approx 30\%$; and $50 < w_L < 80\%$. Using representative values of $\phi' = 32^\circ$ and $I_R = 75$, Figure 14 shows the good fitting observed for the SCE-CSSM expressions for cone, piezocone, and flat dilatometer by adopting an equivalent prestress of $\Delta\sigma'_p = 30$ kPa.

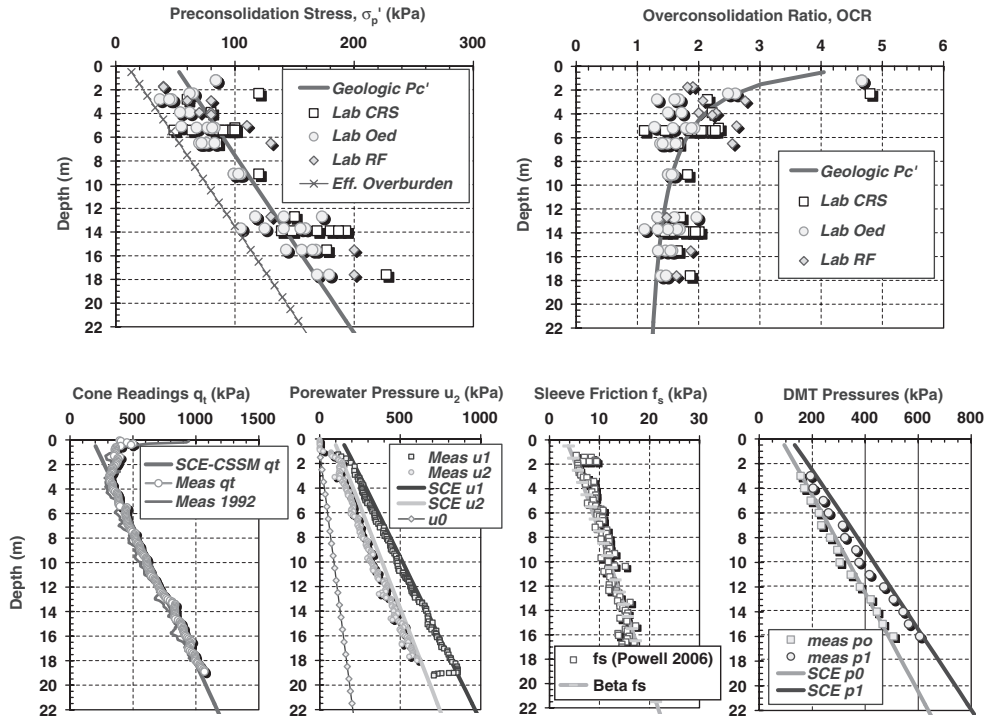


Figure 13. Fitted OCR, CPT, CPTU, & DMT profiles for soft Bothkennar clay (data from Hight et al. 2003).

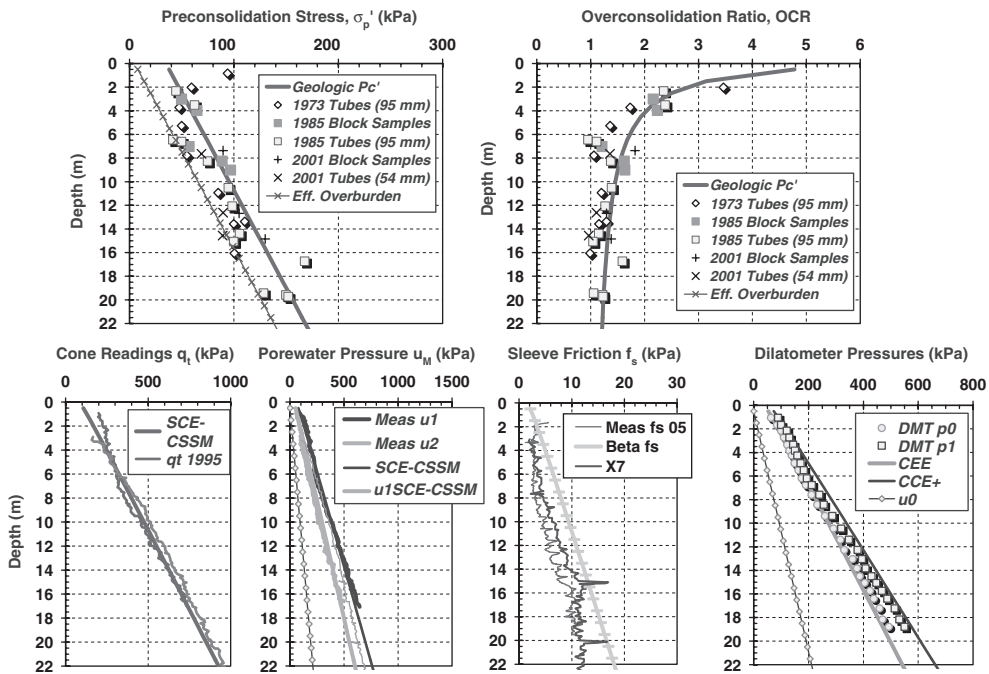


Figure 14. Fitted OCR, CPT, CPTU, and DMT profiles for soft Onsoy clay (data from Lunne et al. 2003).

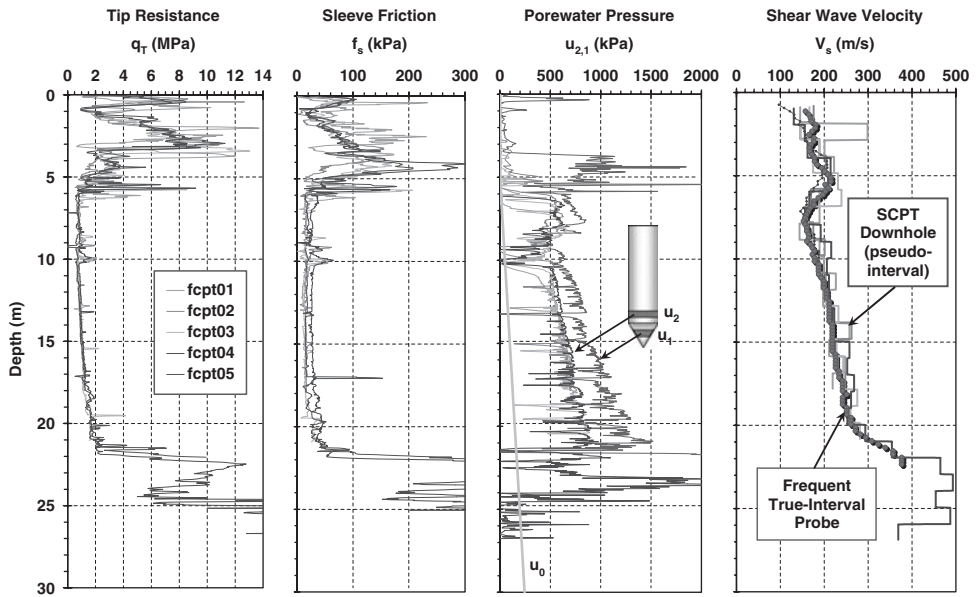


Figure 15. Summary of SCPTU soundings and frequent V_s profiles at Ford Center, Evanston, Illinois.

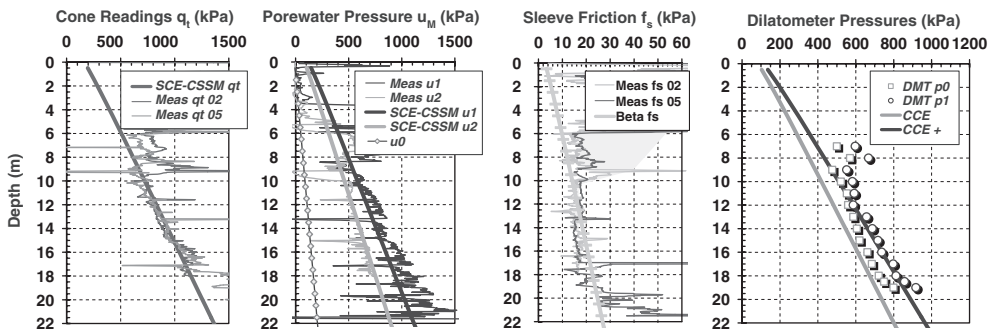


Figure 16. Fitted OCR, CPT, CPTU, and DMT profiles for soft clay at Ford Center, Illinois.

Soft freshwater clay deposits underlie the *Ford Design Center* (Blackburn, et al. 2005; Mayne 2006a) which is located near the national geotechnical experimentation site (NGES) on the campus of Northwestern University in Evanston, Illinois (Finno, 1989; Finno, et al. 2000). The author's CPT crew at Georgia Tech (GT) conducted CPTUs and DMTs at both the Ford site and nearby NGES in 2003 for the National Science Foundation (NSF). At both sites, an upper sand fill overlies soft clays that are lightly overconsolidated with $1.2 < \text{OCRs} < 2$. Results from the 5 SCPTUs, one DMT, and a special frequent-interval downhole-type V_s probing (Mayne 2005) are presented in Figure 15. These soundings delineate the soft clays between 5 and 22 m depths. The forward fitting of CPT and DMT data are presented in Figure 16 using a prestress $\Delta\sigma'_p = 55$ kPa, representative $\phi' = 28^\circ$ (Finno 1989), and rather moderate high value of $I_R = 300$.

To illustrate the approach applicability for overconsolidated materials, two additional case studies from the author's files have been included here.

The stiff clays at the *Baton Rouge* site are Pleistocene age materials extending from 5 to over 40 meters thick (Chen & Mayne, 1994). In-situ test profiles are presented in Figure 17. These deltaic clays are fissured and desiccated. They are geologically & geographically related to the Beaumont clays of the Houston NGES (e.g., O'Neill, 2000). Mean values ($\pm$ one S.D.) from lab index testing gave the water content $w_n = 34 \pm 12\%$; liquid limit $w_L = 60 \pm 20\%$, and plasticity index, $PI = 33 \pm 13\%$. Seven CPTs and one DMT sounding were performed using the cone truck at the Louisiana Transportation Research

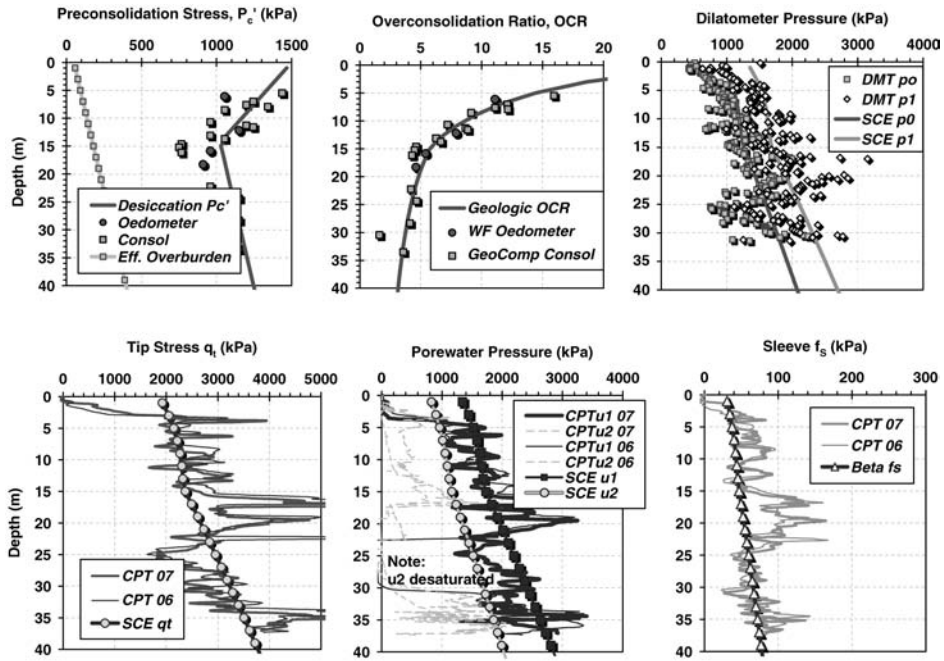


Figure 17. Fitted OCR, DMT, CPT and CPTU profiles for stiff clay in Baton Rouge, Louisiana.

Center (LTRC) by the author under an NSF grant. The results of the piezocone type soundings were obtained using a Fugro triple-element penetrometer that showed good quality data at the midface (u_1) porewater pressure position, but irregular results at the shoulder (u_2) and behind sleeve (u_3) locations, probably due to use of water as the saturating fluid rather than the now-preferred glycerine (Mayne et al. 1995). The overconsolidation difference (OCD), as termed by Locat et al. (2003), is a quasi- prestress ($\Delta\sigma'_p$) that apparently varies with depth in the stiff Baton Rouge clays. Alternatively, a constant quasi-preconsolidation stress on the order of 1 MPa gives similar results (Chen & Mayne 1994). A series of 9 triaxial tests of the CIUC type determined $c' = 0$ and $\phi' = 28.5^\circ$ for these materials. Using these values together with $I_R = 85$ gave the profiles of OCR, q_t , u_1 , u_2 , f_s , and DMT p_0 and p_1 for this site (Figure 16). It can be seen that the general magnitudes for CPTU and DMT readings are in line, yet the desaturated shoulder piezocone elements fall below the expected values, for reasons given above. Additional “squiggly lines” and variances in the measured in-situ profiles reflect other influences, including the macrofabric of fissures and a more complex stress history (as these are Pleistocene age materials), as well as variable sand fractions. Nevertheless, the fitted profiles are notably within the realm of measured resistances that are considerably higher than the four previously-shown soft clays at Bothkennar, Onsøy, Sarapui, and Ford sites.

The stiff *Cooper Marl* lies beneath Charleston, South Carolina, and serves to support most of the large building, dock, and bridge structures in the region. This material appears as a sandy calcareous clay of Oligocene age, with a very high calcium carbonate content between 60 to 80% (Camp, et al. 2002). Representative mean values of index parameters include: $w_n = 48\%$, $w_L = 78\%$, and $PI = 38\%$. Typical SPT-N values in this material are in the range of 12 to 16 bpf. A representative SCPTU is given in Mayne (2005). Separate sets of triaxial testing on undisturbed samples of the calcareous clay by various geotechnical firms for the newly-opened Arthur Ravenel Bridge in 2005 and 15-year old Mark Clark Bridge show consistently high effective stress friction angles $40^\circ \leq \phi' \leq 45^\circ$ for the Cooper Marl. The material has been preconsolidated by erosion and groundwater changes, as well as added structionation due to the calcium carbonate chemistry. Using an $OCD = 480$ kPa, representative $\phi' = 44^\circ$, and very high $I_R = 750$, derived profiles of OCR, CPT, CPTU, and DMT resistances are given in Figure 18.

2.9 First-order OCRs from piezocone and dilatometer

The aforementioned hybrid SCE-CSSM expressions can be used to help ascertain OCR and/or OCD profiles in clay deposits, requiring input values of effective friction angle, compressibility indices,

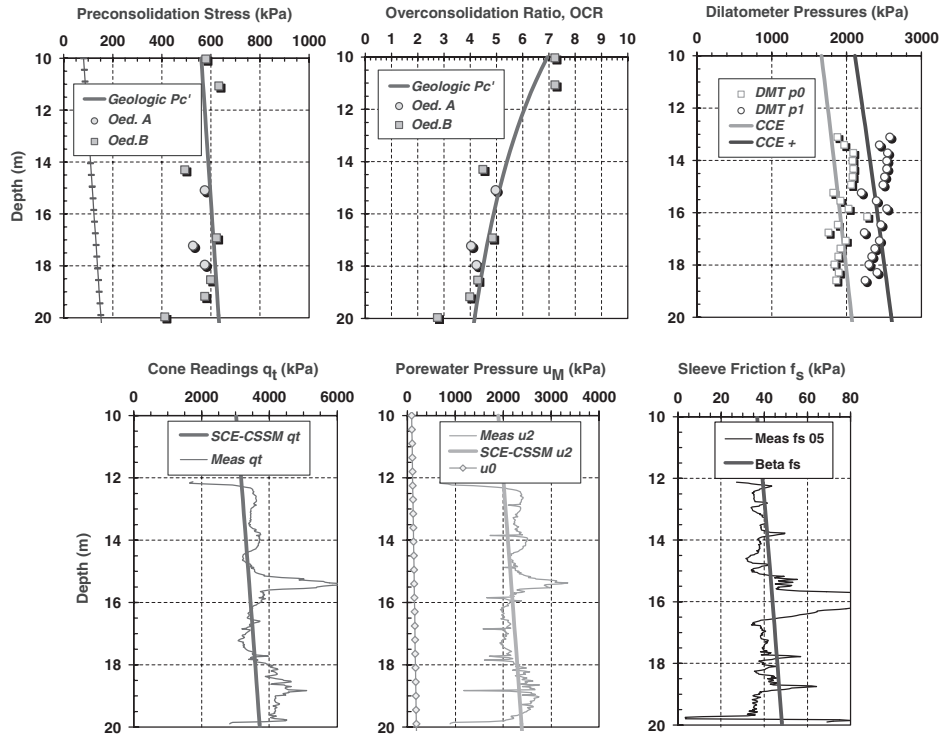


Figure 18. Fitted OCR, DMT, CPT, and CPTU profiles for stiff calcareous Cooper Marl, SC.

and rigidity indices. A simplified approach utilizes representative values (e.g., $\phi' = 30^\circ$, $\Lambda = 0.80$, and $I_R = 100$) to obtain first-order expressions for cone and piezocone penetration in clays (Mayne 2005) that have also been validated by statistical regression analyses of databases in intact soft to firm to stiff clays (Kulhawy & Mayne 1990):

$$\sigma_p' \approx 0.33 (q_t - \sigma_{v0}) \quad (12a)$$

$$\sigma_p' \approx 0.47 (u_1 - u_0) \quad (12b)$$

$$\sigma_p' \approx 0.53 (u_2 - u_0) \quad (12c)$$

$$\sigma_p' \approx 0.60 (q_t - u_2) \quad (12d)$$

Independent studies by Demers & Leroueil (2002) confirmed the applicability of (12a) for 22 clay sites in eastern Canada.

Similarly, expressions for first-order DMT evaluations of preconsolidation stresses in intact clays have been developed (Mayne, 2001):

$$\sigma_p' \approx 0.51(p_0 - u_0) \quad (13)$$

For fissured clays, the above relationships underpredict the preconsolidation stresses because the additional macrofabric of cracks & fissures tends to open up during the advancement of the in-situ probe. In contrast, in the benchmark lab test that defines σ_p' (i.e., the one-dimensional consolidation test), the fissures continually close up during constrained compression. Thus, the coefficient terms for fissured geomaterials in the above expressions could be as much as double those shown for intact clays (Mayne & Bachus, 1989).

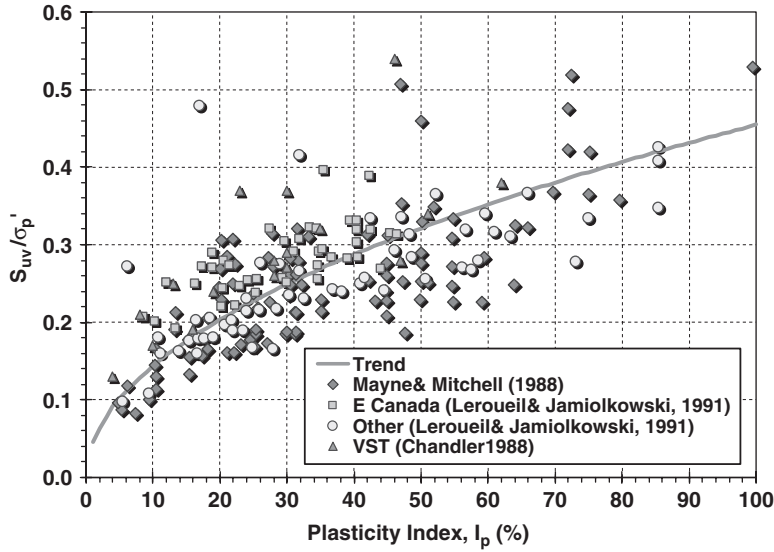


Figure 19. Evaluation of clay preconsolidation from vane strength and plasticity index.

2.10 Other in-situ tests in clays

It is also possible to infer σ'_p profiles in clays from other in-situ tests (SPT, VST, PMT, T-bar, and S-wave velocity). For the vane, the relationship depends upon the clay plasticity (Mayne & Mitchell, 1988), as shown in Figure 19.

The trend for the VST data is represented by:

$$\sigma'_p \approx \frac{22 \cdot s_{uv}}{\sqrt{PI}} \quad (14)$$

For the standard penetration test (SPT), the measured N-value should be corrected to an energy efficiency for the standard-of-practice (60% in the USA), designated N_{60} (e.g., Skempton 1986). In firm to stiff clays which are not highly sensitive nor structured, the approximation is (Kulhawy & Mayne, 1990):

$$\sigma'_p \approx 0.47 N_{60} \cdot \sigma_{atm} \quad (15)$$

where $\sigma_{atm} = 1 \text{ atmosphere} = 1 \text{ bar} = 100 \text{ kPa} \approx 1 \text{ tsf}$.

For the pressuremeter test, a cavity expansion formulation can be implemented to give (Mayne & Bachus, 1989):

$$\sigma'_p \approx 0.5 \cdot s_{uPMT} \cdot \ln(I_R) \quad (16)$$

where $I_R = G/s_{uPMT}$ is the operational value from the PMT. As seen in Figure 20, this is only to be used as a rough guide as the PMT is hardly utilized as an index test for profiling. Data are from self-boring type PMT collected by Mayne & Kulhawy (1990). For the PMT, the value of I_R is generally higher than that corresponding to the large strain I_R value appropriate for the penetration-type probes.

The relatively recent development of full-flow penetrometers have emerged to address the very soft clays found offshore at the murky mudline region (Watson et al. 1998). These offer 3 advantages over classical CPT. First, in lieu of a conventional conical point with 60-degree angle, the penetrometer can be fitted with a larger head-piece (ball, T-bar, or plate) at the front-end to increase resolution of the electronic load cell (i.e., generally 100-cm² area vs. standard 10-cm² with cone). On the second point, the correction of porewater pressures on unequal areas becomes less significant (q_{Tcorr}) because the measured force is much larger. Thirdly, since the soil is assumed to flow around the head, the tip stress is used directly (as compared with net resistance for the CPT: $q_t - \sigma_{vo}$), thus avoiding necessity and uncertainty in evaluation of the overburden stress at each depth. For very soft to soft clays and silts, a similar development in the direct assessment of σ'_p can be made by compilation of a database at sites

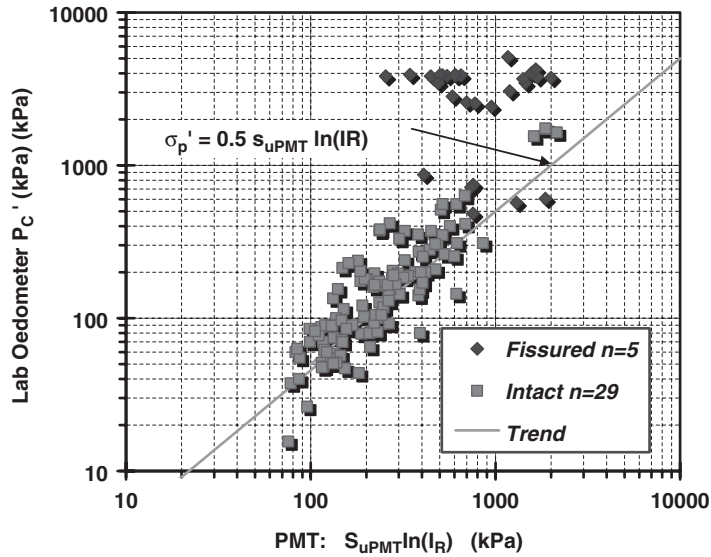


Figure 20. Trend between preconsolidation stress and PMT-calculated Δu in clays.

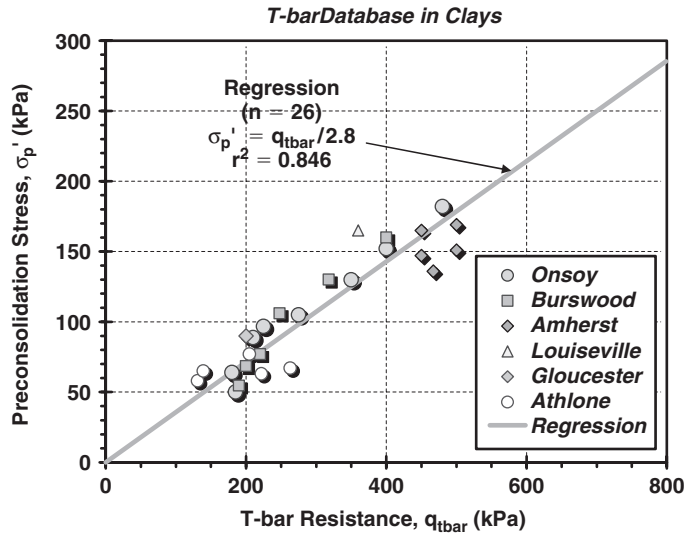


Figure 21. Trend between preconsolidation stress and T-bar resistance in very soft clays.

with known stress history profiles. As such, T-bar penetration data from Onsøy, Norway (Lunne, et al. 2005), Gloucester, Ontario (Yafate & DeJong 2005), Athlone, Ireland (Long & Gudjonsson 2004), Burswood, Australia (Chung, 2005), Amherst, Massachusetts (DeJong et al. 2004), and Louiseville, Quebec (Yafate & DeJong, 2005) are collected in Figure 21 showing:

$$\sigma_p' \approx 0.357 q_{TBar} \quad (17)$$

having a statistical coefficient of determination $r^2 = 0.846$ ($n = 26$).

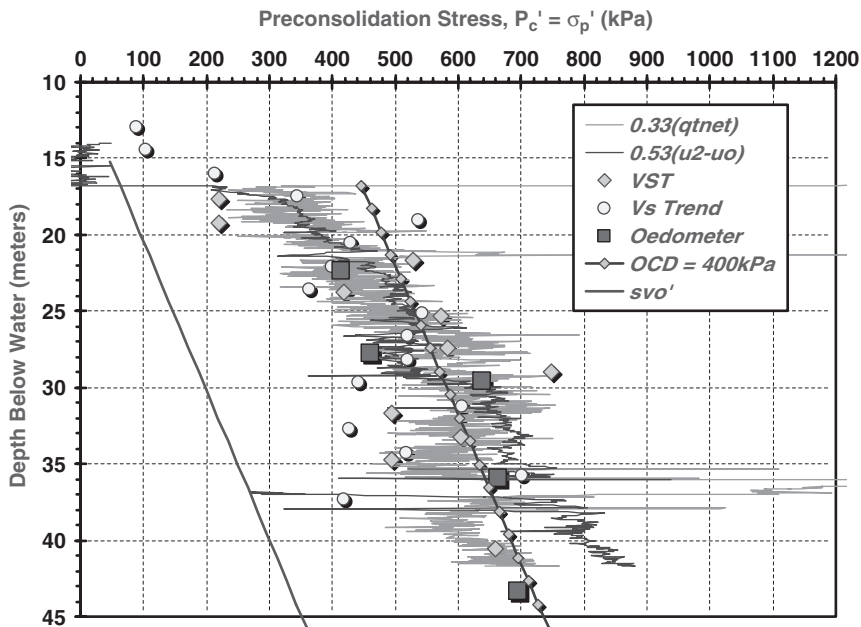


Figure 22. Combined in-situ and lab data for preconsolidation profile in Bootlegger Cove clay.

Finally, the results of shear wave velocity measurements (primarily downhole data) have been used to obtain a first-order evaluation of σ_p' in clays (Mayne, Robertson, & Lunne, 1998):

$$\sigma_p' \approx 0.107 V_s^{1.47} \quad (18)$$

where the preconsolidation stress is stated in units of kPa and shear wave velocity input in units of meters/sec ($n = 262$; $r^2 = 0.823$). This may prove valuable when using non-invasive types of geophysics in clay geologies, such as the surface wave measurements by SASW, CSW, and MASW.

For the above dataset, the coefficient of determination increased ($r^2 = 0.917$) if both net cone resistance and shear wave were included, giving:

$$\sigma_p' = (q_t - \sigma_{vo})^{0.702} (V_s/64)^{0.751} \quad (19)$$

thus optimized by use of the seismic piezocone, since both parameters are determined during the same sounding. In independent validations, Jamiolkowski & Pepe (2001) used (19) with success in Pancone clay at the Pisa tower site.

2.11 Combinative in-situ data

In the interpretation of in-situ tests, there is always uncertainty in the application of available empirical, analytical, and/or numerical methods to a particular clay formation. Thus, it behooves the site characterizer to use a combination of different readings and measurements taken at the site and compare their results for agreement and conformity. Else, convince the client to ante up more funds to conduct additional sampling and testing at the site. By using different test data together, the best estimate profile of stress history can be obtained.

An example of combining in-situ test data is shown below in Figure 22 for the offshore characterization of the lower two facies of the Bootlegger Cove Formation (BCF) at the Port of Anchorage expansion (Mayne & Pearce 2005). A jackup platform (SeaCore) was used for the drilling, sampling, and in-situ testing, since high-to-low tidal variations are 10 m twice a day. The results of cone tip stress, piezocone porewater pressures, vane shear, shear wave velocity, and lab oedometer tests can be collected together to assess the σ_p' profile in the clay. The preconsolidation stress then serves as a common denominator for all test data interpretation in the Bootlegger Cove clays. The approach here for the BCF clay shows the overconsolidated nature in consistent manner with an assumed simple erosional loss or OCD = 400 kPa. Actual inspection of Fig. 22 shows a more complex stress history, with perhaps

sequential layers of different equivalent prestresses, such as $OCD = 320$ kPa from mudline level at 13 to 29 m, $OCD = 460$ kPa from 29 to 37 m, followed by $OCD = 390$ from 37 to 45 m. Below depths of 38 m, the Δu relationship shows a higher interpretation of σ'_p than the net cone resistance value, perhaps signifying a more sensitive facies in the lower part of the clay.

If classical "clay" methods of interpreting undrained shear strengths directly from the lab and field tests were applied instead, significant scatter plots would result, as s_u exhibits many faces and modes. In concept, σ_p is uniquely defined as the yield point on the 1-D consolidation curve, corresponding to the top of the yield surface in 3-D $q - p$ space. If one desires an undrained shear strength value, then application of (2) provides this in a consistent CSSM framework for a representative DSS mode (Trak et al. 1980; Jamiolkowski et al. 1985), else the user can buy into a constitutive soil model to provide other required modes (e.g., CAUC, PSE), such as discussed elsewhere (Ohta et al. 1985; Wroth & Houlsby 1985; Kulhawy & Mayne 1990; Whittle 1993), or by use of empirical methods based on clay plasticity (e.g., Ladd, 1991).

2.12 Stress-strain-strength representation for clays

The stress-strain-strength response of clays begins at the initial small-strain stiffness, represented by the maximum shear modulus: $G_0 = G_{\max} = \rho_T V_s^2$. The equivalent elastic Young's modulus is also available from: $E = 2G(1 + \nu)$, where $\nu' = 0.2$ applies for drained and $\nu_u = 0.5$ for undrained conditions.

The direct simple shear (DSS) gives the best suited mode for stress-strain-strength response for three reasons: (1) *shearing* is the primary mechanism involved in stability & deformation problems, as well as soil-structure interaction situations; (2) DSS data appear much less affected by sample disturbance than the more commonly-available triaxial compression modes (e.g., Lacasse, et al. 1985); and (3) the DSS stress-strain-strength curve is close to the overall average of compression, shear, and extension modes when strain compatibility considerations are made (Ladd, 1991). As such, from the DSS plots of shear stress vs. shear strain (τ vs. γ_s), as shown in Fig. 7, we may define the following: (a) the shear strength is the maximum shear stress ($\tau_{\max}$), (b) the slope is the shear modulus ($G = \tau/\gamma_s$), (c) the initial stiffness is given by the small-strain shear modulus $G_0 = G_{\max}$; (d) the ratio of shear modulus to shear strength is the rigidity index ($I_R = G/\tau_{\max}$); and (e) the strain at failure $\gamma_f = 1/I_R$. For drained loading, the shear strength is given simply by $\tau_{\max} = c' + \sigma'_{vo} \tan \phi'$, with $c' = 0$ applicable in most cases. For saturated soils, the condition for undrained loading is given as $\tau_{\max} = c_u = s_u$, termed the undrained shear strength (per equation 2). At the onset of loading in the field, the soil does not yet know which stress path will be pursued, as this depends upon the applied rate of loading relative to the permeability of the ground. Consequently, the initial stiffness in the field is $G_{\max}$ for any mode (since Δu will not develop until threshold strains are reached later).

A companion to the DSS is another pure shear mode given by the torsional shear (TS) test. This has an advantage in that the test can begin as a resonant column to directly determine the small-strain stiffness ($G_0 = G_{\max}$), then proceed into the intermediate- and large-strain regions to evaluate shear moduli and shear strength. The $G/G_{\max}$ curves can be presented in terms of logarithm of shear strain (γ_s), as discussed by Jardine et al. (1986) and Atkinson (2000), or alternatively in terms of mobilized shear stress ($\tau/\tau_{\max}$), as discussed by Tatsuoka & Shibuya (1992), Fahey & Carter (1993), and LoPresti et al. (1998). With the local strain measurements made on the midsection of triaxial specimens, similar curves of modulus reduction with mobilized deviator stress have been developed (e.g., Tatsuoka & Shibuya, 1992). In terms of fitting stress-strain data, $G/G_{\max}$ vs. mobilized stress level ($\tau/\tau_{\max}$) plots are visually biased towards the intermediate- to large-strain regions of the soil response. In contrast, $G/G_{\max}$ vs. $\log \gamma_s$ curves tend to accentuate the small- to intermediate-strain range. The ratio ($G/G_{\max}$ or G/G_0) is a reduction factor to apply to the maximum shear modulus, depending on current loading conditions. The mobilized shear stress is analogous to the reciprocal of the factor of safety ($\tau/\tau_{\max} = 1/FS$).

A selection of modulus reduction curves, represented by the ratio ($G/G_{\max}$), has been collected from monotonic TS test performed on different materials (Mayne, 2005). The results are presented in Figure 23, where $G = \tau/\gamma = \text{secant shear modulus}$. Additional monotonic static loading test data from special triaxial tests with internal local strain measurements are also included. Here, an assumed constant ν has been applied and the conversion: $E = 2G(1 + \nu)$ to permit $E/E_{\max}$ vs. $q/q_{\max}$. Undrained tests are shown by solid dots and drained tests are indicated by open symbols. In general, the clays were tested under undrained loading (except Pisa), and the sands were tested under drained shearing conditions (except Kentucky clayey sand). Similar curve trends are noted for both drainage conditions (undrained and drained) for both clays and sands.

Nonlinear representation of the stiffness has been a major focus of the recent series of conferences on the theme *Deformation Characteristics of Geomaterials*. A number of different mathematical expressions

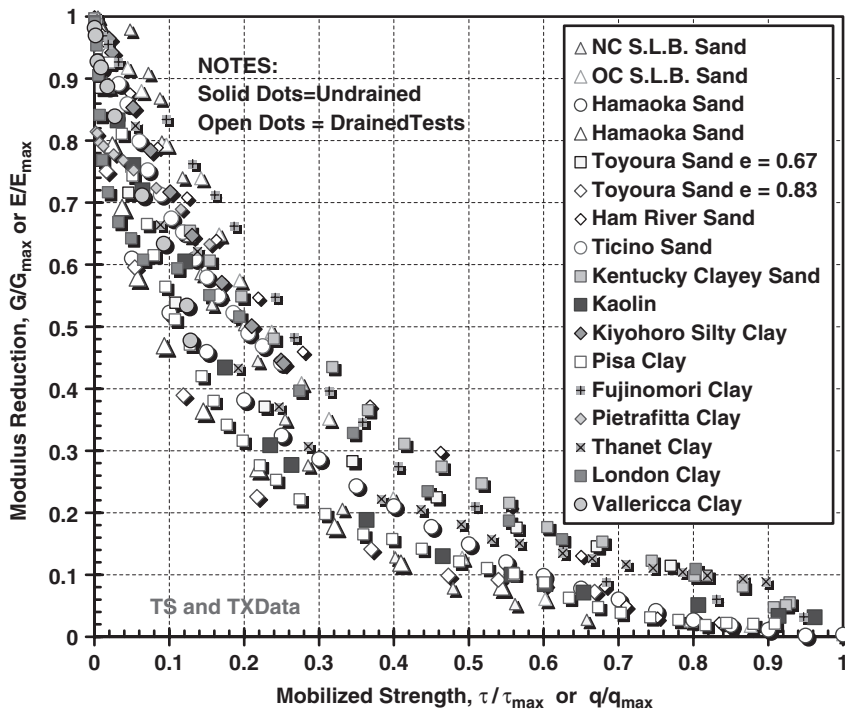


Figure 23. Modulus reduction from TS and TX data on clays and sands vs. mobilized strength.

can be adopted to produce closed-form stress-strain-strength curves (e.g., LoPresti et al. 1998; Shibuya, et al. 2001; Santos & Correia, 2001; Tatsuoka et al. 2003; Jardine et al. 2005). For example, a simple hyperbola requires only two parameter constants for a nonlinear stress-strain representation, however, it can only fit one region of the strain range (small or intermediate or large), as discussed by Tatsuoka & Shibuya (1992). Thus other expressions have been sought. One example of the derived modulus reduction curves for a modified hyperbola (Fahey & Carter, 1993) is shown in Figure 24, with $f = 1$ and $0.2 < g < 0.4$ encompassing much of the TS and TX data shown previously. The stress-strain-strength curve can be defined by the shear strength ($\tau_{\max}$), initial shear modulus ($G_0 = G_{\max}$), and adopted exponent ($g \approx 0.3 \pm 0.1$ for “hourglass sands” and “vanilla clays”). Then the shear stress (τ) and secant shear modulus (G) can be determined at any load level ($\tau/\tau_{\max} = 1/\text{FS}$):

$$\tau = G \cdot \gamma_s \quad (20)$$

$$G = G_{\max} \cdot [1 - (\tau/\tau_{\max})^g] \quad (21)$$

Alternatively, a logarithmic function (Puzrin & Burland, 1996) can be employed with a simplified version relying on a normalized strain parameter (x_L) in the range: $20 < x_L < 50$ for the TS data set, where $x_L = G_{\max}/G_{\min}$, and $G_{\min} = \tau_{\max}/\gamma_f$. This has the advantage that all parameters have a recognized physical significance. Calibration fittings for soils using the log function are discussed by Elhakim & Mayne (2000).

An illustrative fitting for both approaches is presented for San Francisco Bay Mud, where field and lab data are reported by Pestana, et al. (2002). Results of SCPTu sounding at the California site are given in Figure 25. The cone tip stress, excess porewater pressures, and/or effective cone resistance can be used in (12) to obtain the estimated profile of σ'_p and OCR, which in turn, are utilized in (2) to evaluate the DSS mode for undrained shear strength. The stress-strain-strength curves obtained using $s_{u\text{DSS}} = 21.8$ kPa and $G_0 = 10.3$ MPa are shown in Figure 26 for the modified hyperbola ($g = 0.35$) and log function ($x_L = 35$) with good agreement for both cases compared with measured DSS data on undisturbed samples taken from 7.3 m depth.

A unified constitutive model for sand and clay (Pestana & Whittle, 1999) provides theoretical relationships for $G/G_{\max}$ reduction with log shear strain that are found in good comparison with cyclic data

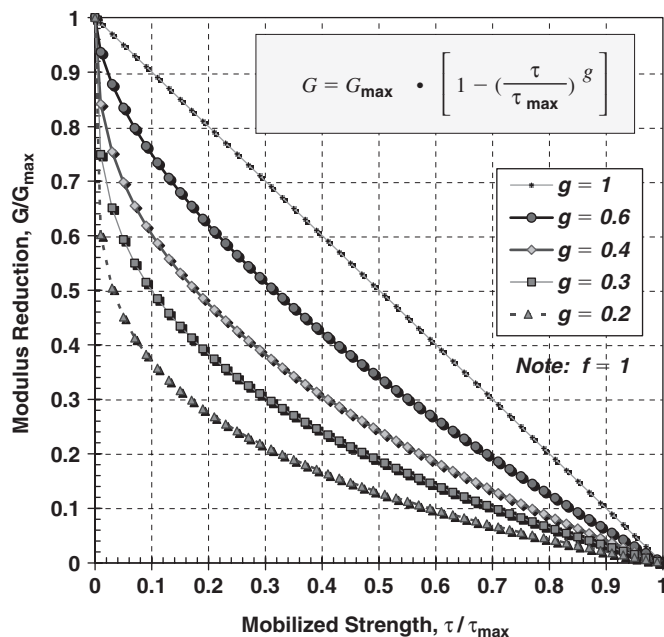


Figure 24. Modulus reduction vs. mobilized strength using modified hyperbola (after Fahey 1998).

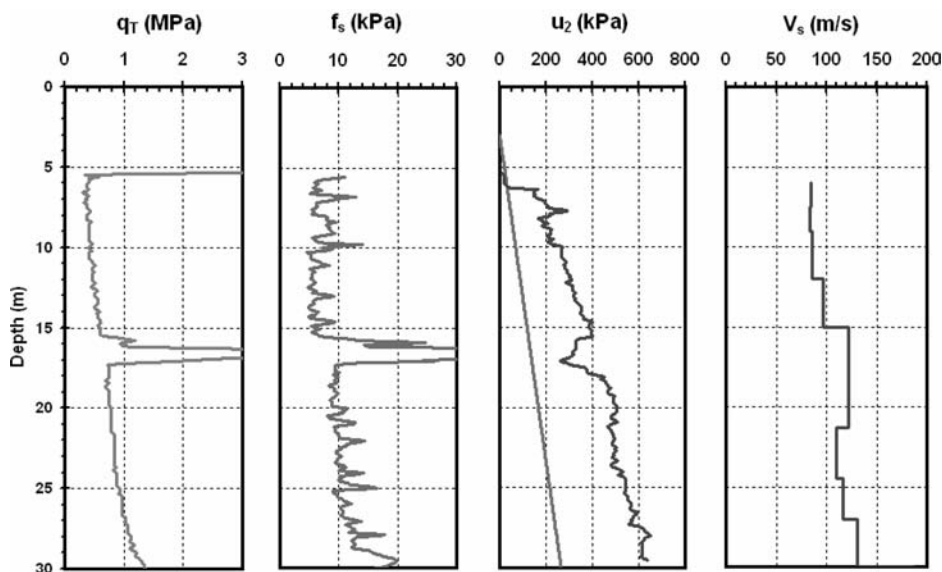


Figure 25. Results of SCPTU in San Francisco Bay Mud (data from Pestana et al. 2002).

from resonant column tests summarized by Vucetic & Dobry (1991). Careful studies by Shibuya, et al. (1997) and Yamashita, et al. (2001) have shown that differences in monotonic and cyclic reduction curves are predominantly explained by strain rate effects, thus the author suggests that monotonic curves be used as a basis for all other curves, that afterwards, can be adjusted for strain rate and/or uppers to form dynamic curves, as necessary.

In terms of in-situ testing, the use of mechanical wave geophysics (CHT, DHT, SASW, SL, SCPT, SDMT) is significant because the initial shear modulus can be defined. Since the SCPTu provides

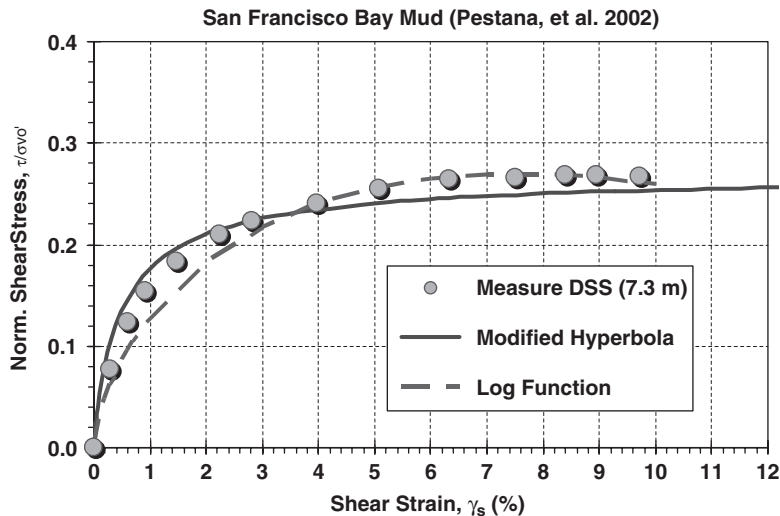


Figure 26. Measured and fitted DSS stress-strain-strength curves for San Francisco Bay Mud.

information on both the initial stiffness ($G_{\max}$) at small-strains, as well as shear strength ($\tau_{\max}$ from s_u and/or ϕ') at large-strains, the difficult question is how to determine stiffnesses at intermediate strain levels on a site-specific basis, without assuming values. One prospect is the cone pressuremeter (Schnaid & Houlsby, 1992; Ghionna, et al. 1995) with a geophone setup to produce a seismic piezocone pressuremeter (SCPMtu). Another approach would be the utilization of paired side-by-side in-situ tests, such as companion sets of SCPTu and DMT (Tanaka & Tanaka, 1998), or the commercially-available SDMT (e.g., Foti, et al. 2006).

2.13 Strain rate effects

Soils exhibit strain rate effects and these can be addressed by carefully-controlled lab tests (Tat-suoka, et al. 1997; Shibuya et al. 1997). Commonly, the triaxial test has been utilized to investigate effects of strain rate effects on undrained stress-strain behavior, with stiffness and strength both increasing with rate of loading (e.g., Sheahan et al. 1996). Yet, considerable research has also shown that consolidation results (both e - $\log \sigma'_v$ curves and yield stress, σ'_p) shift up with increasing strain rate, particularly in clays (Leroueil & Hight 2003). Consequently, the results of in-situ tests in the field are affected by strain rate. Of additional complication in the field is that drainage can occur during testing.

For vane shear tests, data were compiled by Peuchen & Mayne (2006) from 27 field sites and 7 lab series on clays. Two common means to show rate effects in soils include (1) time-to-failure, t_f , and (2) power function format (Soga & Mitchell, 1996). For the VST data, Figure 27 shows the effects of time-to-failure on normalized undrained shear strength (s_{uv}/s_{uv0}). These appear fairly consistent over a wide range of loading conditions, particularly from 0.01 minutes to 10,000 minutes. Here, the strain rate effect on undrained vane strength may be seen to increase on the order of 10 to 14% per log cycle which is comparable to that observed for laboratory test data (e.g., Kulhawy & Mayne, 1990; Leroueil & Hight, 2003). At very fast rates ($t_f < 0.01$ min), apparent dynamic effects may also occur, although those data are biased towards the small lab size equipment and use of remolded and synthetic clays.

Using the alternate plotting of normalized vane strength vs. rotation rate, the data are presented in Figure 28. The power law format shows that the exponent term is generally between 0.05 and 0.10, in agreement with lab triaxial and consolidation data (Soga & Mitchell, 1996).

For laboratory triaxial testing, truly undrained conditions can be maintained by locking off the drainage ports. In this case, as the shearing rates decrease to very slow values, the undrained shear strength will level off to a minimum value. Thus, in lieu of the semi-log function depicted in Figure 27 to represent strain rate effects on s_u , Randolph (2004) suggested an hyperbolic sine function that captures this facet.

In the field, if the rate of loading decreases significantly, then partial to full drainage can occur during in-situ testing, since there is no control on drainage. Lunne et al. (1997) present a table of CPT studies that look at rate effects and these can be consulted to further investigate rate effects concerning penetration

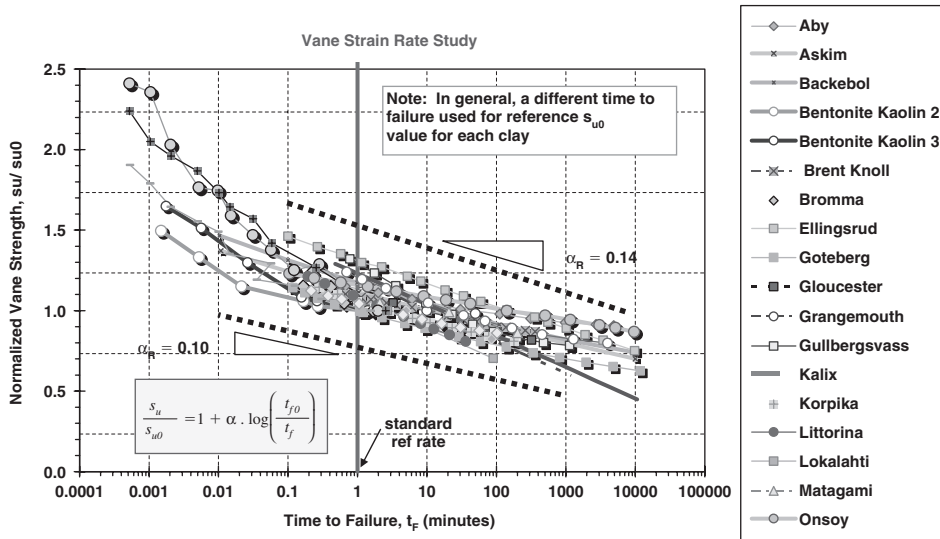


Figure 27. Variation of vane shear strengths with time-to-failure (Peuchen & Mayne 2006).

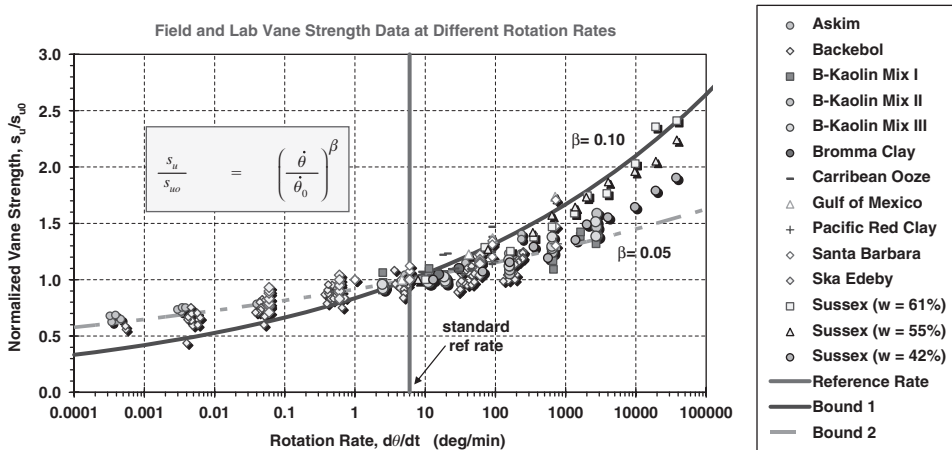


Figure 28. Trend of vane shear strength with increasing rate of rotation (Peuchen & Mayne 2006).

testing. Randolph (2004) has proposed the addition of “twitch testing” to help evaluate strain rate and drainage effects, as discussed below in Section 2.16.

2.14 Dissipation testing

In fine-grained soils, undrained loading is a transient case and represents but a particular stress path at constant volume ($\Delta V/V_0 = 0$) within an infinite number of possible stress paths in effective $q - p'$ space. Given sufficient time, the temporary perturbation caused by in-situ testing will seek equilibrium and the dissipation of excess porewater pressures. The rate at which equilibrium is achieved is governed by the coefficient of permeability (k) of the medium, or alternatively by the coefficient of consolidation (c_{vh}):

$$c_{vh} = \frac{k \cdot D'}{\gamma_w} \quad (22)$$

where $D' =$ constrained modulus and $\gamma_w =$ unit weight of water. For most natural clays, the horizontal permeability is only 5 to 10% higher than the vertical value (Leroueil & Hight, 2003).

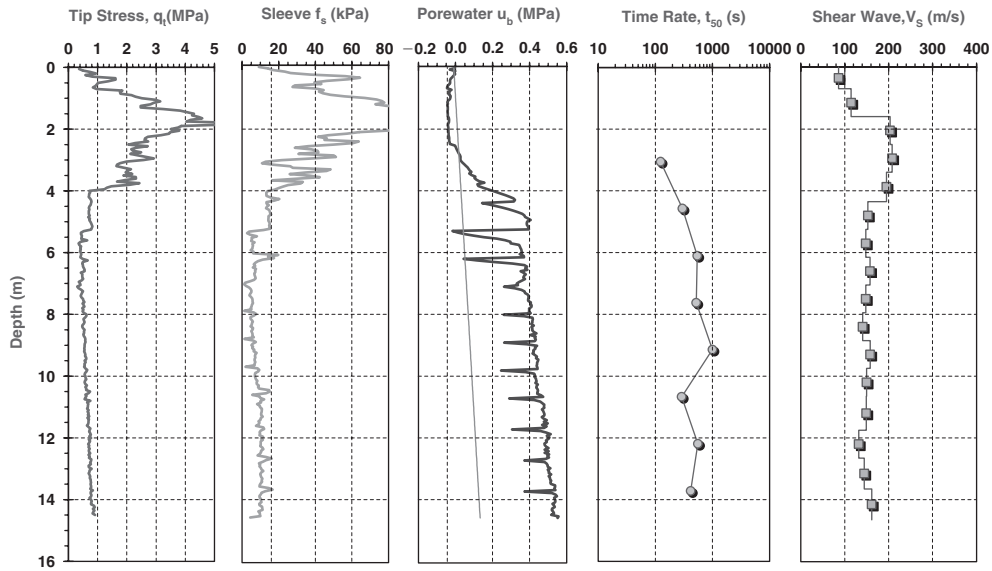


Figure 29. Seismic piezocone test with dissipations (SCPTu) at the Amherst soft clay test site.

Since piezocone tests (CPTu) directly measure penetration porewater pressures, the practical emphasis is to utilize dissipation readings to evaluate c_{vh} in soils, although procedures have also been developed for PMT (Fahey & Goh 1995; Clarke & Gambin 1998) and DMT (Robertson et al. 1998; Marchetti et al. 2001). The most popular CPTu method at present is the strain path method (SPM) solution of Houlsby & Teh (1988), although other available procedures are discussed by Jamiolkowski et al. (1985), Senneset et al. (1988, 1989), Danzinger et al. (1997), Burns & Mayne (1998, 2002), and others.

In terms of calibrating an approach, a fairly comprehensive study between lab c_v values and piezocone c_h values in clays and silts was reported by Robertson, et al. (1992). Assumptions were made between the ratio of horizontal to vertical permeability to address possible issues of anisotropy during interpretation. The study compared laboratory-determined results with the SPM solution (Teh & Houlsby 1991) using data from type 1 piezocones (22 sites) and type 2 piezocones (23 sites), as well as 8 sites where backcalculated field values of c_{vh} were obtained from full-scale loadings.

With the SPM approach in practice, it is common to use only the measured time to reach 50% consolidation, designated t_{50} . As such, if dissipation tests are carried out at select depth intervals during field testing, a fairly optimized data collection is achieved by the SCPTu since five measurements of soil behavior are captured in that single sounding: q_t , f_s , u_b , t_{50} , and V_s . The results of a (composite) SCPTu in the soft varved clays at the NGES in Amherst, Massachusetts are depicted in Figure 29. Here the results of a GT sounding are augmented with data from a separate series of dissipations conducted by DeGroot & Lutenege (1994).

In lieu of the focus on a single point corresponding to 50% degree of consolidation (U_{50}), other degrees of dissipation can be considered, or even better, an entire range of consolidation data points starting from penetration through the entire testing time. As such, Houlsby & Teh (1988) provided time factors for a range of porewater pressure dissipations. The degree of excess porewater pressure dissipation can be defined by: $U^* = \Delta u / \Delta u_i$, where Δu_i = initial value during penetration. The modified time factor T^* is defined by:

$$T^* = \frac{c_{vh} \cdot t}{a^2 \cdot \sqrt{I_R}} \quad (23)$$

where t = corresponding measured time during dissipation and a = probe radius. The SPM solutions between U^* and T^* for midface u_1 and shoulder u_2 piezo-elements are shown in Figures 30 and 31, respectively. These can be conveniently represented using approximate algorithms as shown, thus offering a means to implement matching data on a spreadsheet.

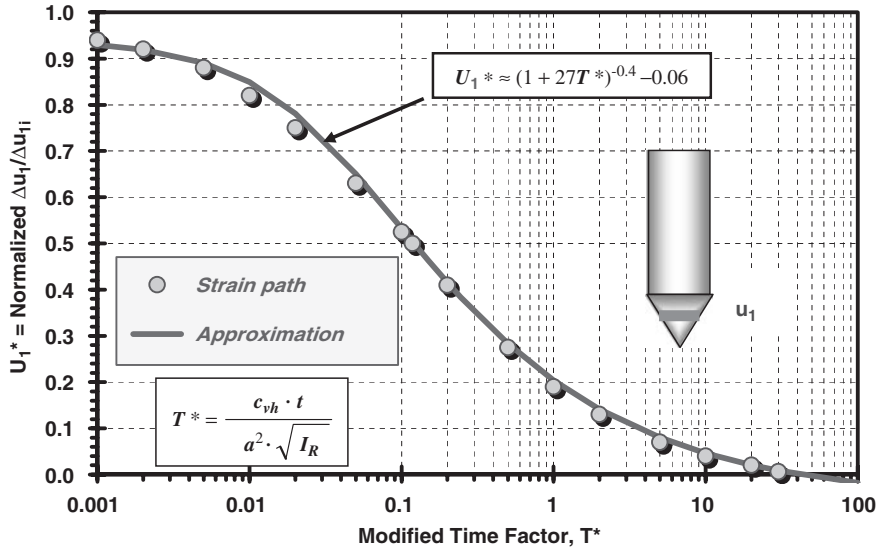


Figure 30. Dissipation response for type 1 midface piezo-elements (SPM solution of Teh & Houlsby 1991).

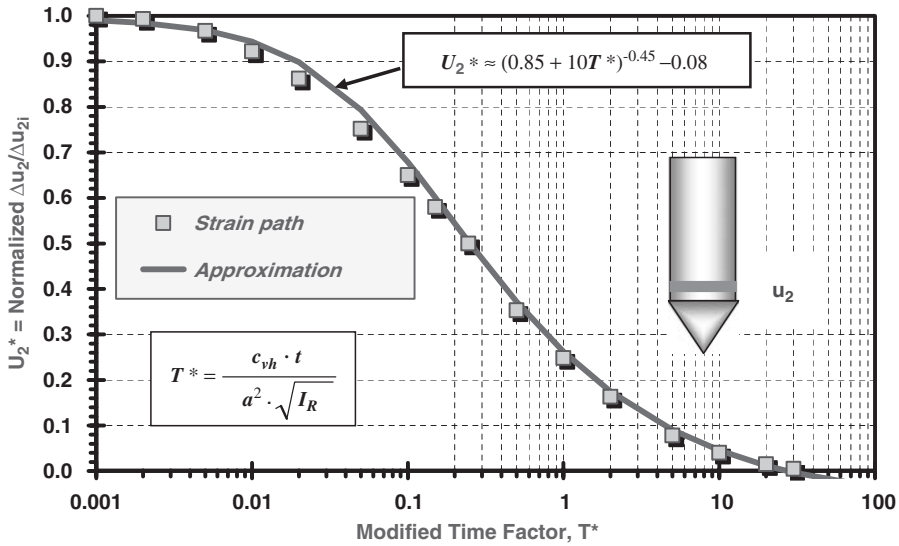


Figure 31. Dissipation response for type 2 shoulder piezo-elements (SPM solution of Teh & Houlsby 1991).

To use the method, the following procedures are suggested:

- Assume a range of values for T^* starting from 10^{-5} to say 100. Note: Choose intermediate log values at the 1's, 2's, and 5's places.
- Calculate the corresponding U^* dissipations per the approximate expressions given in Figures 30 or 31, as appropriate.
- Calculate the actual time by rearrangement of (23) to obtain: $t = T^* a^2 I_R^{0.5} / c_{vh}$. Plot U^* vs. t .
- Conduct parametric analyses by varying c_{vh} and/or I_R until a fitting with the measured data is achieved.

For the I_R value, a value may have already been attained from the fitting of penetration readings. With the aforementioned procedure, the results of dissipation tests from the Bothkennar soft clay site have been

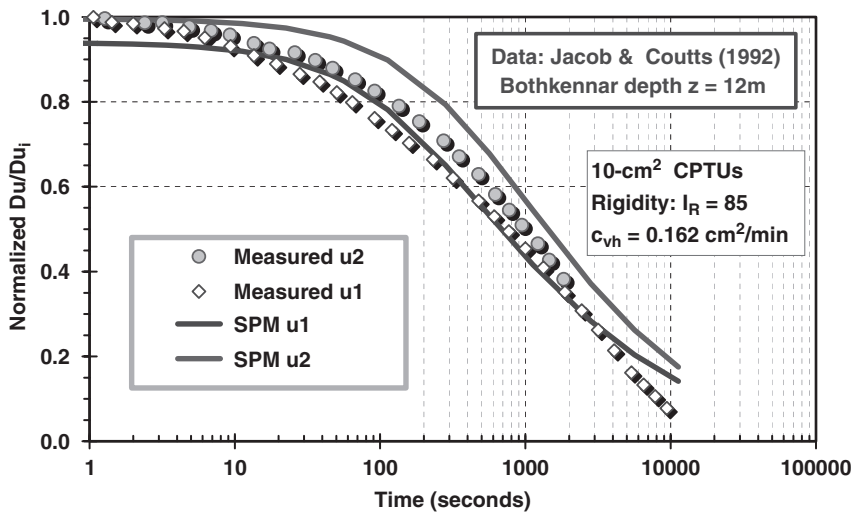


Figure 32. Forward SPM predictions and measured dissipations at Bothkennar site, UK.

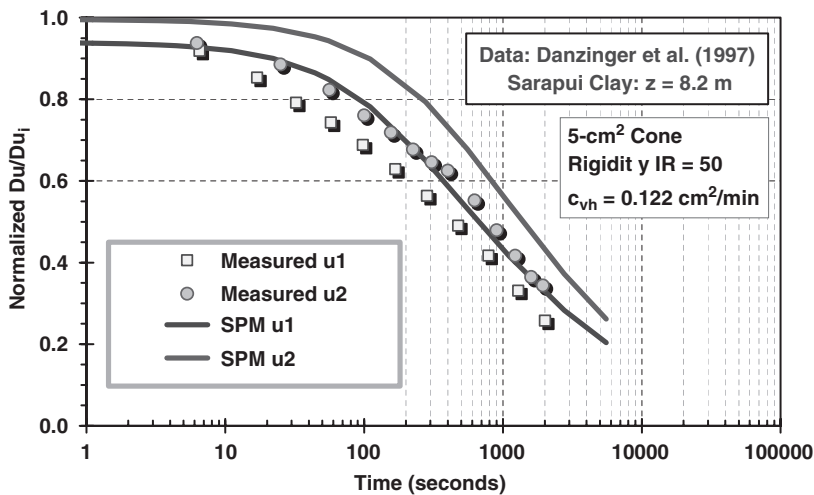


Figure 33. Forward SPM predictions and measured dissipations at Sarapui site, Rio.

forward fitted using the same corresponding value of rigidity index given in Table 1 ($I_R = 85$) and used in Figure 13. Different types of 10-cm² cones were used ($a = 1.78$ cm). The value of c_{vh} used to drive the analysis was chosen from full-scale footing load test performance reported by Jardine et al. (1995). Both short-term and long-term footing behavior was monitored and a backfigured $c_{vh} = 8.5$ m²/year (0.0027 cm²/s) was determined. Figure 32 presents the dissipation fitting for both type 1 and 2 piezo-dissipation data reported by Jacob & Coutts (1992) at a depth of 12 m at the site. Overall, there is good agreement between the SPM procedures and measured piezocone data.

A similar set of calculations have been carried out for the soft Sarapui clay in Brazil. Data from 5-cm² quad-element piezocone dissipation tests have been reported by Danzinger et al. (1997) and Schnaid et al. (1997). The $I_R = 50$ from Table 1 has been used and a backfigured field value from embankment monitoring gave $c_{vh} = 0.122$ cm²/min (Robertson, et al. 1992). The SPM predictions are shown for midface and shoulder elements in Figure 33, indicating some underpredictions in rate of decay. A better match is obtained with $c_{vh} = 0.2$ cm²/min.

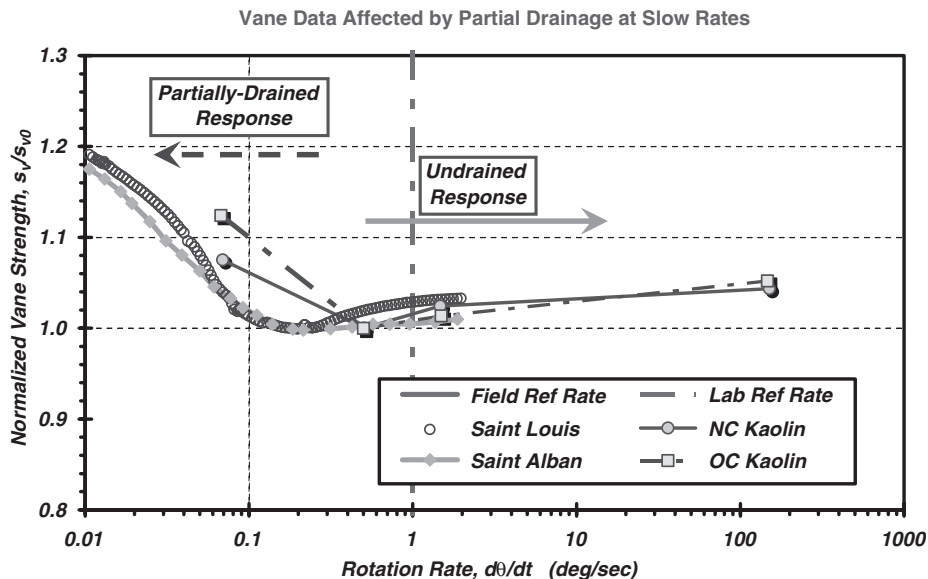


Figure 34. Threshold velocity on undrained vane response an onset of partial drainage.

2.15 Partial drainage considerations

If in-situ tests in fine-grained soils are conducted slower than standard rates, a partially-drained or “semi-undrained” condition may prevail whereby dissipation occurs during the penetration or probing. This may actually be the common condition in silty soils because of the relative fast rates of in-situ testing relative to the moderate to high permeability of these materials.

Randolph (2004) has suggested special series of “twitch tests” in clays whereby a variable rate of testing by the penetration probe provides data on both strain rate effects and partial drainage. His work has involved piezocones and full-flow probes, such as the T-bar. A dimensionless velocity term can be defined by:

$$V = v \cdot d / c_{vh} \quad (24)$$

where v = velocity and d = probe diameter. The dimensionless velocity V helps to quantify the regions that are governed by (a) undrained behavior (and strain rate), (b) semi-drained, and (c) fully-drained behavior. This would appear a valuable means to fit within the CSSM framework and allow a rational assessment of intermediate stress paths occurring between fully undrained and fully drained. The demarcation between the undrained region for VST response and partially-drained behavior can be seen for field test data from Canada and lab series on kaolinitic clay using a miniature vane in Figure 34 (Peuchen & Mayne 2006).

In the above discussions of dissipation and partial drainage, monotonic decay of induced porewater pressures has been addressed. Yet, in many overconsolidated materials, a dilatory response can occur (e.g., Sully & Campanella, 1994; Burns & Mayne, 1998). Also, an incompatibility exists for the examples shown as the penetration and porewater data were represented by cavity expansion but the monotonic porewater decay addressed using strain path. What is needed is a consistent framework in SCE and/or SPM, else in finite elements, finite differences, and/or discrete elements.

3 IN-SITU TESTS IN SANDS

Available methods for evaluating the mechanical properties of sands have developed from an assortment of empirical-statistical, analytical, theoretical, and numerical simulation approaches. Their validity has mostly been established on the basis of reconstituted samples tested in the lab, on either small-size triaxial specimens ($25 < D < 75$ mm) or large calibration chamber tests (CCT with $0.9 < D < 2.5$ m), or combination of both. The CCTs allow for the complete insertion of the in-situ device. However,

the calibration chamber results must be corrected for the limited size chambers, since flexible-walled CCTs will under-register the measured penetration resistances compared to far-field conditions, whilst rigid-walled CCTs will over-register the readings.

Of recent vintage, special freezing methods have been developed to obtain field undisturbed samples of sands from the same medium as the in-situ testing locations (e.g., Hoffman et al. 1996). The expensive and time-consuming process requires a slow moving unidirectional freezing front. This is done so that the sand fabric and natural structure are not destroyed during the transformation of water to ice at $T < 0^\circ\text{C}$ that is accompanied by a volumetric increase of around 8.5%.

In this section, a review of CPT interpretative methods derived from corrected CCTs for clean quartz sands and siliceous sands (comparable parts of quartz & feldspar particles) is given, with a few well-chosen case study examples, followed by a short supplement of SPT-based methods. Then, a special dataset based on undisturbed sands is presented to evaluate select interpretative procedures. The emphasis in this paper is on CPT, SPT, and V_s methods, supplemented with DMT and PMT data where available.

3.1 Cone penetration in clean sands

Using a large dataset of 702 CCTs conducted on 26 different clean quartz to siliceous sands with appropriate boundary size corrections applied to the measured CPT tip resistances, the derived relationship for obtaining the effective stress friction angle was (Kulhawy & Mayne 1990):

$$\phi' = 17.6^\circ + 11.0^\circ \log(q_{t1}) \quad (25)$$

where $q_{t1} = (q_t / \sigma_{atm}) / (\sigma'_{vo} / \sigma_{atm})^{0.5}$ is the stress-normalized cone tip stress and $\sigma_{atm} = 1 \text{ bar} = 100 \text{ kPa}$. The normalization to square root of effective overburden stress likely helps to account for sand compressibility and grain crushing effects, to some degree.

The effective lateral stress applied in chamber tests affects the cone tip stress, more so than the effective vertical stress. Thus, the applied consolidation state used in the CCTs is given by both the lateral stress coefficient ($K_0 \approx K_c = \sigma'_{hc} / \sigma'_{vc}$) and overconsolidation ratio (OCR) that has been related to the measured q_t values via statistical analyses of the CCT data (Mayne 2005):

$$K_0 = 0.192 \left(\frac{q_t}{\sigma_{atm}} \right)^{0.22} \left(\frac{\sigma'_{vo}}{\sigma_{atm}} \right)^{-0.31} OCR^{0.27} \quad (26)$$

The use of the above expression is rather limited, since the field evaluation of stress history of natural sands is often quite elusive. However, if an a priori relationship between K_0 and OCR can be established, the procedure can move forward. For instance, a methodology based on a combining of (4) with (26) is of the form (Mayne, 2001):

$$OCR = \left[\frac{0.192 \cdot (q_t / \sigma_{atm})^{0.22}}{(1 - \sin \phi') \cdot (\sigma'_{vo} / \sigma_{atm})^{0.31}} \right]^{\left(\frac{1}{\sin \phi' - 0.27} \right)} \quad (27)$$

where the apparent preconsolidation stress of the sand can be calculated from:

$$\sigma'_p = OCR \cdot \sigma'_{vo} \quad (28)$$

3.2 CPT case study examples

Two case studies can be used to illustrate the evaluation of strength and stress history of natural sands: (1) nearly NC Po River Sand, Italy (Ghionna et al. 1995); and (2) OC glacial sand near Stockholm, Sweden (Dahlberg 1974). In both cases, the stress history is relatively well-known based on geologic setting and rather extensive lab & field testing which have been carried out.

The *Po River Sand* is a relatively thick deposit of clean to slightly silty sand that has been subjected to a great number of in-situ tests, including SPT, CPT, CPMT, PMT, SBP, DMT, and V_s (Jamiolkowski, et al. 1985; Bruzzi, et al. 1986; Ghionna et al. 1995). The sand is lightly overconsolidated due to groundwater fluctuations and ageing. Special sampling of occasional clay lenses of geologically-related materials were captured and tested in oedometers to quantify the stress history. Additional data on the stress state (i.e., K_0) was obtained from the independent pressuremeter testing (SBPMT). Figure 35 shows the application of the aforementioned relationships in evaluating the respective profiles of measured q_t

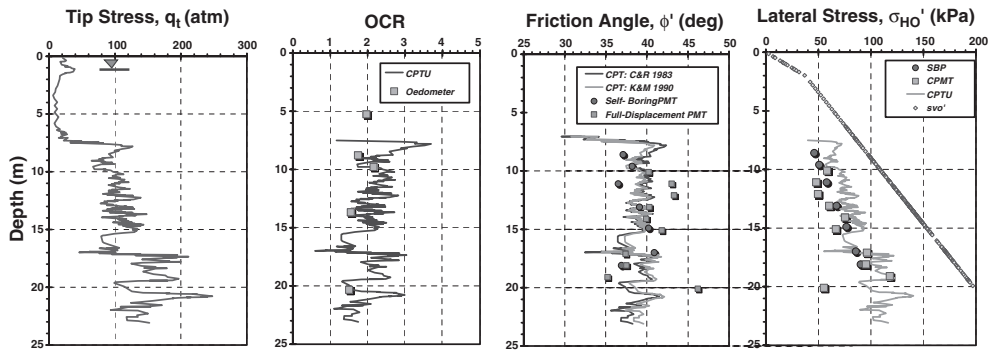


Figure 35. Profiles of CPT q_t , OCR, ϕ' , and σ'_{ho} for Po River Sand (data from Ghionna et al. 1995).

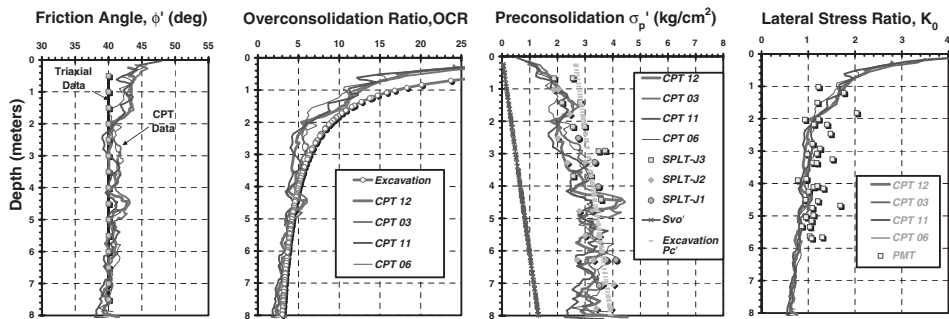


Figure 36. CPT-interpreted profiles of ϕ' , OCR, and K_0 in Stockholm Sand (data from Dahlberg 1974).

and interpreted ϕ' , OCR, and $\sigma'_{ho} = K_0 \cdot \sigma'_{vo}$ in Po River sand. Reasonable values of $1.5 < OCR < 2.5$ are obtained from the CPT data relative to the few oedometric results available. Results from self-boring type (SBP) and full-displacement type (CPMT) pressuremeter testing have been used to independently assess the effective ϕ' and K_0 conditions, where again the CPT values are seen to be in general accord.

For the *Stockholm Sand*, a Holocene deposit of clean glacial medium-coarse sand was quarried for use in construction, having an initial 24 m thickness overlying bedrock. After the upper 16 m was removed, series of in-situ testing (SPT, CPT, PMT, SPLT) were performed, in addition to special balloon density tests in trenches. Groundwater lies at the base of the sand just above bedrock. Index parameters of the sand include: mean grain size ($0.7 < D_{50} < 1.1$ mm); uniformity coefficient ($2.2 < UC < 3$), mean density $\rho_T = 1.67$ g/cc, and average $D_R \approx 60\%$. Results of the screw plate load tests (SPLT) were used by Dahlberg (1974) to interpret the preconsolidation stresses in the sand, very comparable to the known geologic values from mechanical overburden removal ($OCD = \Delta\sigma'_v$). Results of the lift-off pressures from PMTs gave corresponding K_0 values in the sand. Using results of 4 Borros electric CPTs at the site, Figure 45 shows the derived profiles of ϕ' , OCR, and σ'_p in Stockholm sand. The CPT-interpreted ϕ' agrees well with triaxial tests on reconstituted samples. The stress history from CPT is consistent with the OCD and SPLT data, with additional support on K_0 given by PMT results.

3.3 SPT penetration in clean sands

For the standard penetration test (SPT), empirical methods have been produced for assessing the strength and stress history of clean quartz to siliceous sands. Using results based on triaxial tests on frozen sand specimens, the effective stress friction angle can be obtained from the energy-corrected and stress-normalized N-value from the SPT (Hatanaka & Uchida 1996; Mayne et al. 2002):

$$\phi' = 20^\circ + \sqrt{15.4 \cdot (N_1)_{60}} \quad (29)$$

$$\text{where } (N_1)_{60} = N_{60}/(\sigma'_{vo}/\sigma_{atm})^{0.5}.$$

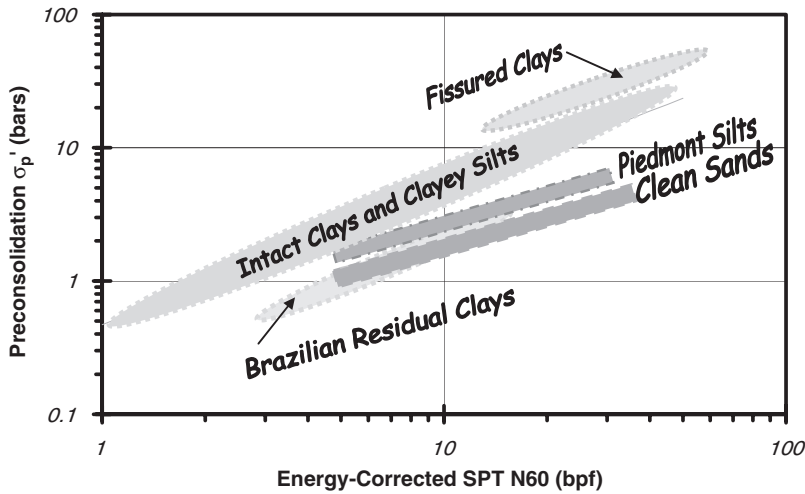


Figure 37. Apparent preconsolidation stress vs. N_{60} for soils (after Mayne 1992).

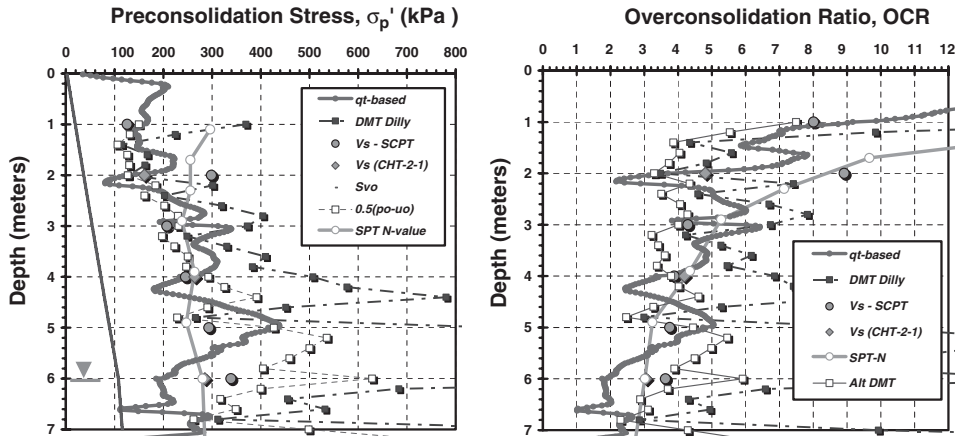


Figure 38. Combinative in-situ test interpretations of stress history at College Station sand site, Texas.

The apparent preconsolidation stress in clean sands can be estimated from the relationship given in Figure 37. This can be combined with (15) in generalized form (Mayne, 1992):

$$\sigma'_p \approx 0.47 (N_{60})^m \cdot \sigma_{\text{atm}} \quad (30)$$

where $m = 0.6$ for clean quartzitic sands, 0.8 for silty sands to sandy silts (e.g., Piedmont), and $m = 1.0$ for intact “vanilla” clays to clayey silts.

3.4 Application case study to TAMU sand

A US national geotechnical experimentation site (NGES) has been established in Eocene age sand near Texas A&M University (Briaud 2000). Here the 10-m thick sand is relatively clean in the upper 4 or 5 meters, becoming slightly to somewhat more silty and clayey with depth. Extensive series on in-situ tests have been carried out at the site, including SPT (with and without energy measurements), CPTU, SCPT, DMT, PMT, and CHT, as well as full-scale load tests on pilings and footings (Briaud & Gibbens, 1994). The CPT evaluation gives an operational friction angle $\phi' = 39^\circ$ in the sands (Mayne 1994). The combined use of the SPT and CPT procedures for assessing profiles of preconsolidation stress and OCR at the NGES sand site located in College Station, Texas are shown in Figure 38. Both the SPT- and CPT-based profiles

are in general agreement with each other. Also shown is an evaluation using shear wave velocity measurements from the site (both DHT by SCPT and CHT), based on a generalized approach (Mayne 2005):

$$\sigma_p' = 0.101 \cdot \sigma_{atm}^{0.102} \cdot G_{max}^{0.478} \cdot \sigma_{vo}'^{0.420} \quad (31)$$

Available DMT data from the site was input into the DILLY software program and these profiles are also presented, along with values from (13). Unfortunately, no real known OCR profile of this sand is documented, so that validity of these results cannot be checked. It is therefore apparent that significant research is needed in establishing the calibration and documentation of natural sand sites, particularly with respect to stress history considerations.

3.5 CE and CSSM frameworks for sands

Cavity expansion theory can be applied to cone penetration in sands since it represents an inverse problem of pile end bearing capacity (Vesić, 1972, 1977). Sand parameters include the effective friction angle (ϕ'), effective cohesion intercept (c'), rigidity index (I_R), and magnitude of induced volumetric strain ($\Delta V/V_0$). In lieu of the latter parameter, a formulation by Carter et al. (1986) develop a CE solution in terms of the dilatancy angle (ψ'). For the case of $c' = 0$ for these approaches, the normalized tip stress term is q_t/σ_{vo}' and therefore requires additional information in order to assess I_R and either ($\Delta V/V_0$) or ψ' (Mitchell & Keaveny, 1986; Yu 2004). Recently, Mayne (2006) showed that the operational rigidity index, defined as $I_{RR} = 1/[(1/I_R) + (\Delta V/V_0)]$, may be related to the normalized shear modulus (G_0/σ_{vo}').

Critical-state concepts have been used to organize sand strength data together include the effects of state (relative density), confining stress level, mode, and dilatancy, as well as differences in mineralogy (Bolton, 1986). In this regard, the method is hindered somewhat because the penetration test data (i.e., SPT, CPT) are used solely to evaluate the in-place relative density (D_R) while values of the fitting parameters (Q , R , and baseline ϕ'_{cs}) are often assumed (Jamiolkowski, et al. 2001). The advantage is that the CSSM framework does allow variants and adjustments to sands of differing origins and constituencies. Using the Bolton CSSM relationship for peak ϕ' in sands, Salgado et al. (1997a, 1997b, 1998) developed a numerical code (*CONPOINT*) that utilizes cavity expansion theory and initial shear modulus to model cone penetration in sands. This has been extended now to address silty sands within a similar framework (Salgado et al. 2000; Lee et al. 2004).

Been & Jefferies (1985) present an alternate CSSM framework to represent sand behavior. They define the sand state parameter (Ψ) as the vertical difference in void ratio between the initial in-place condition and the critical state line (CSL), termed steady state line ($\Psi = e_0 - e_{cs}$). In this light, Been et al. (1986, 1987) were able to relate CPT resistances to describe sand response ranging from loose contractive to dense dilatant behavior, as well as undrained modes. Series of laboratory triaxial shear tests are needed to define λ (and κ), yet recent efforts by Cho et al. (2006) have related the parameters to simple index tests. The state parameter method has since been implemented into a modeling framework for CPT (Shuttle & Jefferies, 1998). Of particular interest herein, it is possible to show that Ψ is actually just another means to express stress history, and thus related to OCR. In fact, Been et al. (1988) show:

$$\log OCR_p = \log 2^\Lambda + \Psi/(\kappa - \lambda) \quad (32)$$

where OCR_p = overconsolidation ratio in Cambridge $q - p'$ space, $\Lambda = 1 - \kappa/\lambda$, $\lambda = C_c/\ln(10)$ = compression index, and $\kappa \approx C_s/\ln(10)$ = swelling index, with latter two slopes defined in $e - \ln p'$ space (although some confusion here as original reference is defined in terms of log base 10). The state parameter method has since been implemented into a framework for CPT interpretation (Shuttle & Jefferies, 1998) that involves a CSSM constitutive soil model (*NorSand*) that appears to be quite versatile (Jefferies & Shuttle, 2005).

3.6 Undisturbed sand database

Conventional approaches to cross-linking of lab-determined engineering parameters and in-situ field tests have relied upon reconstituted samples, specimens, and chamber deposits. For reconstitution, a number of preparation methods are available, including: vibration, compaction, air and water pluviation, moist tamping, and slurring. The primary assumption is that if the sand is replaced at the same initial density and void ratio in the laboratory, then this will give comparable behavior to the same natural sand that exists in the field. This approach is depicted in Figure 39. It is further assumed that our present practices of assessing the in-place relative density (D_R), unit weight (γ_T), and/or void ratio (e_0) of that

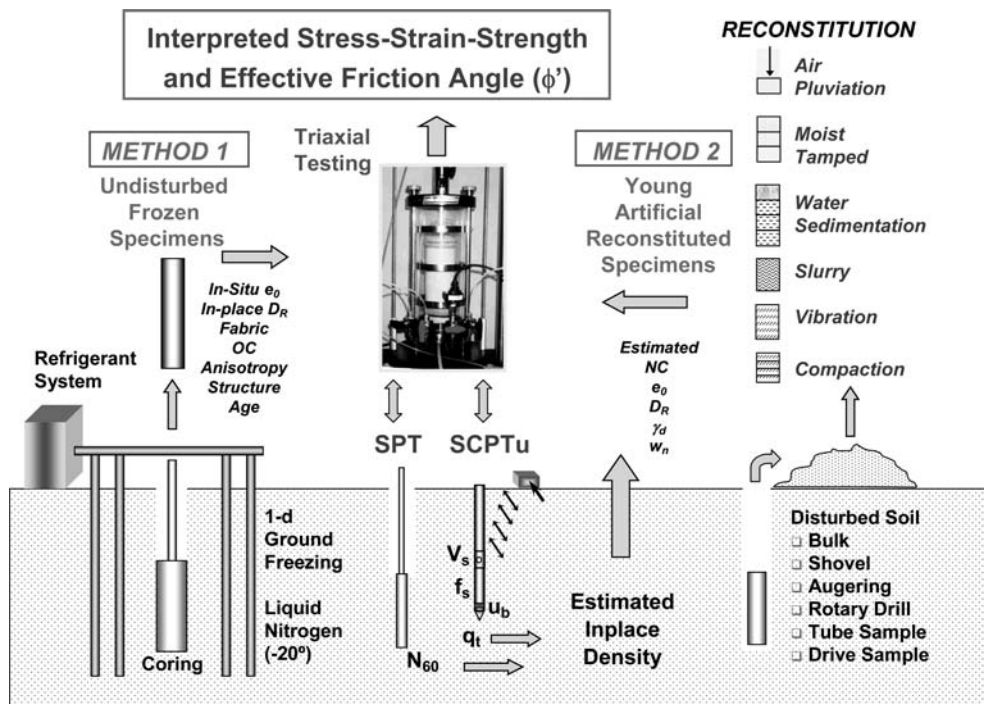


Figure 39. Procedures depicting undisturbed vs. reconstituted methods for calibrating in-situ tests in sands.

sand from in-situ test methods (SPT, CPT, DMT) is relatively reliable. Neither of these assumptions is well-supported, as shown by recent comparisons from triaxial tests on the same sand tested in undisturbed states as well as artificially prepared sand samples by different reconstitutive measures (e.g., Hoeg, et al. 2000; Vaid & Sivathayalan 2000; Jamiolkowski 2001). Available methods to assess D_R from in-situ tests do not appear especially reliable, particularly using penetration type tests including SPT and CPT (e.g., Wride et al. 2000) and DMT (e.g., Jamiolkowski, et al. 2001).

To further check on the validity of these major assumptions, a new elite database on undisturbed clean quartz and siliceous sands was created. A total of 13 sands that were sampled using special techniques (primarily expensive freezing technology) was collected (Mayne 2006b).

The sands are located in Canada (6 sands), Japan (4 sands), Norway (1 site), China (1 site), and Italy (1 site), as listed in Tables 2 and 3 with pertinent index parameters and reference sources of the data. Notably, all four of the Japanese sands (Mimura 2003), two Canadian sands (Robertson et al. 2000), and the Holmen, Norway (Lunne et al. 2003) were discussed at the Singapore workshop. In general, the sands can be considered as clean to slightly dirty sands of quartz, feldspar, and/or other rock mineralogy, excepting two of the Canadian sands derived from mining operations that had more unusual constituents of clay and other mineralogies.

In terms of grain size distributions, the materials include 10 fine sands, 4 medium sands, and one coarse sand (Italy). The sands from Canada were dirty having fines contents: $5 < FC < 15\%$, whereas the other sands were all relatively clean with $FC < 4\%$. Mean values of index parameters (with plus and minus one standard deviation) of these sands indicated: specific gravity ($G_s = 2.66 \pm 0.03$), fines content ($FC = 4.36 \pm 4.49$), particle size ($D_{50} = 0.35 \pm 0.23 \text{ mm}$)¹, and uniformity coefficient ($UC = D_{60}/D_{10} = 2.80 \pm 1.19$). The mean grain size statistics would be better represented by a log normal function.

At all sites, results from SPT and CPT were available, as well as downhole V_s measurements (except the China site). A summary of mean values of the normalized SPT (N_1)₆₀, normalized CPT results

¹ Note: Coarse sand from Italy with $D_{50} = 2 \text{ mm}$ not included. If included, then $D_{50} = 0.46 \pm 0.48 \text{ mm}$.

Table 2. List of undisturbed sands, reference sources of data, origin, and sample type.

Sand name	Country location	Reference source	Type sand	Sampling method
Edo River	Japan	Yamashita et al. (2003); Mimura (2003)	Natural Alluvial	Frozen
Gioia Tauro	Italy	Ghionna & Porcino (2003, 2006) Porcino & Ghionna (2004)	Natural Coarse Sand	Frozen
Highmont Dam	Canada	Wride & Robertson (1999); Robertson et al. (2000)	Tailings	Frozen
Holmen	Norway	Lunne, et al. (1986); Lunne, Long, & Forsberg (2003)	Natural Alluvial	Tube (Frozen)
J-pit	Canada	Robertson, Wride, et al. (2000); Wride & Robertson (1999)	Hydraulic Fill	Frozen
Kidd	Canada	Robertson, Wride, et al. (2000) Wride & Robertson (1999)	Natural Alluvial	Frozen
Kowloon	China	Lee, Shen, Leung, Mitchell (1999)	Hydraulic Fill	Mazier Tube
LL Dam	Canada	Robertson, Wride, et al. (2000)	Tailings	Frozen
Massey	Canada	Robertson, Wride, et al. (2000)	Natural Alluvial	Frozen
Mildred Lake	Canada	Robertson, Wride, et al. (2000); Wride & Robertson (1999)	Hydraulic Fill	Frozen
Natori River	Japan	Matsuo & Tsutsumi (1998); Mimura (2003)	Natural Alluvial	Frozen
Tone River	Japan	Matsuo & Tsutsumi (1998) Mimura (2003)	Natural Alluvial	Frozen
Yodo R. (8)	Japan	Mimura (2003)	Natural Alluvial	Frozen
Yodo R.(10)	Japan	Mimura (2003)	Natural Alluvial	Frozen
Yodo R.(12)	Japan	Mimura (2003)	Natural Alluvial	Frozen

Table 3. Undisturbed sands, mean depths, index parameters, and overburden stress levels.

Sand name	Depth z (m)	GWL (m)	PF (%)	D ₅₀ (mm)	G _s	e _o	e _{max}	e _{min}	DR (%)	σ' _{vo} (atm)
Edo	3.85	2.1	0.42	0.29	2.68	1.043	1.227	0.812	44.3	0.51
Gioia Tauro	3	1.5	0.66	2	2.69	0.589	0.690	0.450	42.0	0.42
Highmont	10	4	10	0.25	2.66	0.825	1.015	0.507	37.4	1.38
Holmen	10	1	2	0.55	2.71	0.724	0.840	0.460	30.5	1.10
J-pit	5	0.5	15	0.17	2.62	0.762	0.986	0.461	42.7	0.55
Kidd	14.5	1.5	<5	0.2	2.72	0.981	1.100	0.700	29.8	1.60
Kowloon	11.8	9	1	0.72	2.63	0.492	0.634	0.328	46.5	1.80
LL Dam	8	2.1	8	0.2	2.66	0.849	1.055	0.544	40.3	1.00
Massey	10.5	1.5	<5	0.2	2.68	0.970	1.100	0.700	32.5	1.20
Mildred L.	32	21	10	0.16	2.66	0.768	0.958	0.522	43.6	5.16
Natori	8.25	2.1	0.23	0.22	2.65	0.857	1.167	0.765	77.2	0.87
Tone	7.3	1.4	3.78	0.18	2.68	0.947	1.330	0.775	69.1	0.84
Yodo	8.1	2.1	1.9	0.32	2.64	0.820	1.054	0.665	60.2	1.02
Yodo	10.9	2.1	0.27	0.82	2.63	0.720	0.883	0.569	51.9	1.23
Yodo	12.7	2.1	2.1	0.62	2.63	0.790	0.921	0.567	37.0	1.43

(Q , F , R_f , q_{t1} , and $\Delta u/\sigma'_{vo}$), and normalized shear wave velocity (V_{s1}) is presented in Table 4, where $(N_1)_{60} = N_{60}/(\sigma'_{vo}/\sigma'_{atm})^{0.5}$, $Q = (q_t - \sigma_{vo})/\sigma'_{vo}$, $F = f_s/(q_t - \sigma_{vo})$, $R_f = f_s/q_t(\%)$, $q_{t1} = q_t/(\sigma'_{vo} \cdot \sigma'_{atm})^{0.5}$, and $V_{s1} = V_s/(\sigma'_{vo}/\sigma'_{atm})^{0.25}$.

An initial check on the data is made by cross-linking the in-situ measurements using prior-available relationships. Figure 40 shows a plot of cone resistance versus the energy-corrected N-values for the sands. A common assumption is that the direct ratio of cone tip resistance (bars) to N-value (bpf) is in the range of $4 \leq q_t/N_{60} \leq 5$ for such materials (e.g., Schmertmann, 1978). For the 15 datapoints considered, the ratio averages $q_t/N_{60} = 4.38$, thus within the expected range. The specific ratio q_t/N_{60} has been shown to partially depend upon the mean grain size of the soil (D_{50}), as well as the percent fines content, FC (e.g., Robertson & Campanella 1983; Kulhawy & Mayne 1990). However, it appears this ratio decreases with low N-values and that a better overall match would be attained using either an intercept, such as $q_t/(N_{60} + n^*)$ with a value $n^* = 6$ bpf, else fitting of a power function format such that $q_{t1} = \alpha^* [(N_1)_{60}]^{\beta^*}$, as suggested by Suzuki, et al. (1998). The minor improved coefficients of determination (r^2) shown in

Table 4. Undisturbed sands, triaxial friction angles, and in-situ SPT, CPT, and DHT data.

Sand	Triaxial type	ϕ' (deg)	SPT (N_1) ₆₀	Q	F	R _f (%)	q _{t1}	$\Delta u/\sigma'_{vo}$	V _{s1} (m/s)
Edo River	CIDC	39.7	22	156	0.73	0.72	111	-0.08	164
Gioia Tauro	CIDC, CIUC, CTX	41.5*	30	279	0.26	0.26	174	-0.26	221
Highmont	CIUC, CKoUC	41.5	5	35	0.38	0.38	44	0.11	141
Holmen	CKoUC, CKoDC	33.2	1	29	0.42	0.40	28	-0.33	157
J-pit	CIUC, CKoUC	32.7	3	31	0.87	0.75	20	0.19	127
Kidd	CIUC, CKoUC	37.3	13	52	0.37	0.36	68	-0.02	177
Kowloon	CIUC	38.1	21	55	1.84	1.80	74	0.07	NA
LL Dam	CIUC, CKoUC	39.1	5	39	0.41	0.39	39	0.01	153
Massey	CIUC, CKoUC	36.7	10	49	0.40	0.38	53	-0.09	168
Mildred Lake	CIUC, CKoUC	39.6	18	32	0.73	0.70	74	0.02	156
Natori R.	CIDC	40.9	50	226	0.30	0.30	207	-0.48	218
Tone R.	CIDC	41.7	32	154	0.24	0.24	147	-0.46	203
Yodo(8)	CIDC	42.4	27	194	0.95	0.94	176	-0.05	197
Yodo(10)	CIDC	38.4	35	103	1.03	1.01	102	-0.19	213
Yodo(12)	CIDC	39.1	31	96	0.89	0.88	106	-0.19	195

*Note: Results above for final q/p' from CTX series; monotonic tests give 37.8°.
NA = not available

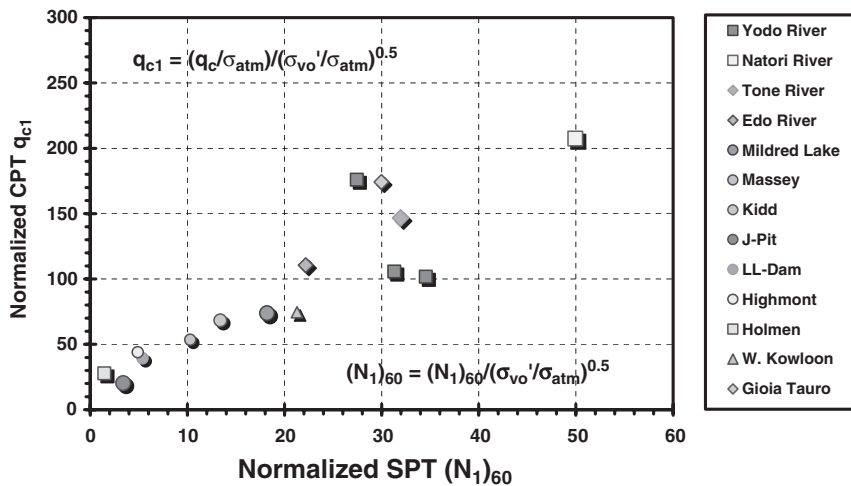


Figure 40. Trend between CPT and SPT data for undisturbed sand database.

Figure 41 lend support to this notion. Alternatively, the procedures for stress normalization of SPT and/or CPT data may be involved in the observed trends (e.g., Boulanger & Idriss 2004; Moss, et al. 2006).

Andrus & Stokoe (2000) present a relationship between the normalized shear wave velocity, $V_{s1} = V_s/(\sigma'_{vo}/\sigma_{atm})^{0.25}$, and the energy-corrected & stress-normalized N-value, $(N_1)_{60}$. The data from the undisturbed sands appear to conform fairly well with this correlation, as shown by Figure 42a. Similarly, Andrus et al. (2004) developed a similar interrelationship between V_{s1} and normalized cone tip stress (q_{t1}), as supported by the dataset from the undisturbed sands (reference Figure 42b). Thus, the compiled in-situ data from SPT, CPT, and V_s appear internally consistent.

All of the undisturbed sands were tested under consolidated triaxial compression tests to determine their respective effective stress friction angles (ϕ'). The measured values are reported in Table 4. This permits an opportunity to check on the validity of selected relationships in evaluating sand strength from available in-situ test data (Mayne 2006b). Using the aforementioned expression between ϕ' and $(N_1)_{60}$ developed by Hatanaka & Uchida (1996), only a fair agreement is noted for the new dataset in

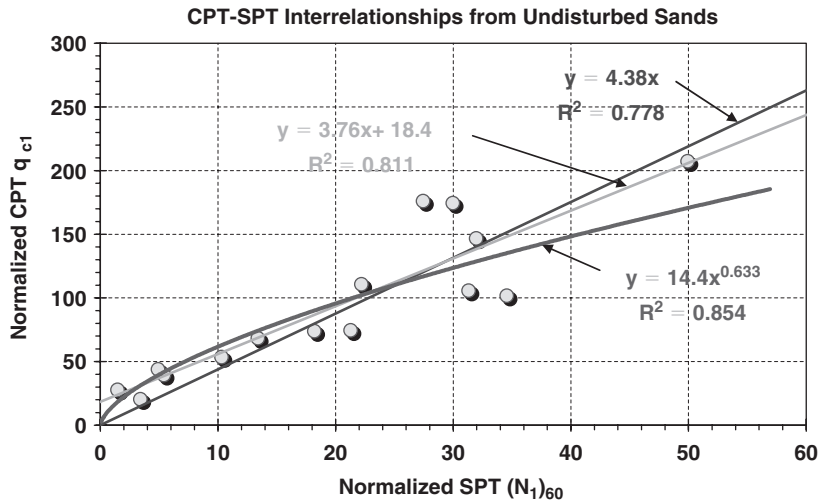


Figure 41. Regression relationships between CPT and SPT for undisturbed sand database.

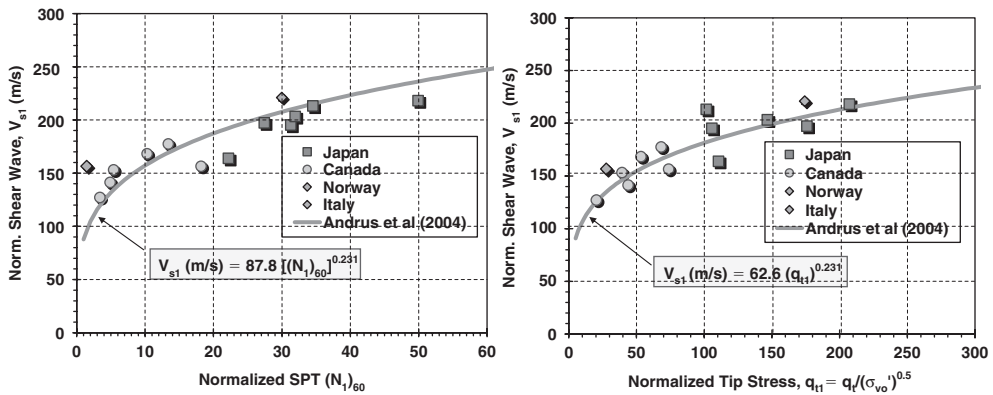


Figure 42. Normalized shear wave velocity vs. (a) SPT $(N_1)_{60}$ and (b) CPT q_{t1} for undisturbed sands.

comparison with their frozen sand samples from additional Japanese test sites (Figure 43). The trend is much flatter for the new data and suggests a relative insensitivity of $(N_1)_{60}$ in accurately estimating effective ϕ' in sands. The friction angles of loose to firm sands at low to medium N-values are significantly underpredicted by the expression. Perhaps, the repeated dynamic nature of the SPT at low N-values is more indicative of an “undrained” type cyclic loading.

The relationship between the triaxial-measured ϕ' of undisturbed sands and normalized cone tip resistance is presented in Figure 44. Here, the CPT proves to be an excellent predictor in representing the drained strength of the sands. The two outliers from LL and Highmont Dams are sands with some unusual mineralogies beyond the normal quartzitic & feldspathic types, exhibiting higher percentages of clay and other minerals (as noted). In fact, if the mica content were also included, LL Dam would have 50% and Highmont would show composition of 20% of minerals other than quartz & feldspar.

Another interesting comparison is the between the apparent OCRs as suggested from the SPT and CPT data, shown in Figure 45. The agreement from these two separate approaches is quite remarkable for all fifteen sand datapoints. Both the SPT and CPT indicate values of OCR < 1 for three sands (Holmen, Highmont, and Mildred Lake), with values up to as high as OCR = 7 for Gioia Tauro sand. Sands with OCRs < 1 imply unstable structure and perhaps this could be used as an indicator of liquefaction potential. The CPT data also infers that the sands at J-pit and LL dam have OCRs ≤ 1 (although SPT results indicate OCR > 1). This is of interest as both LL and Highmont were noted earlier to have some

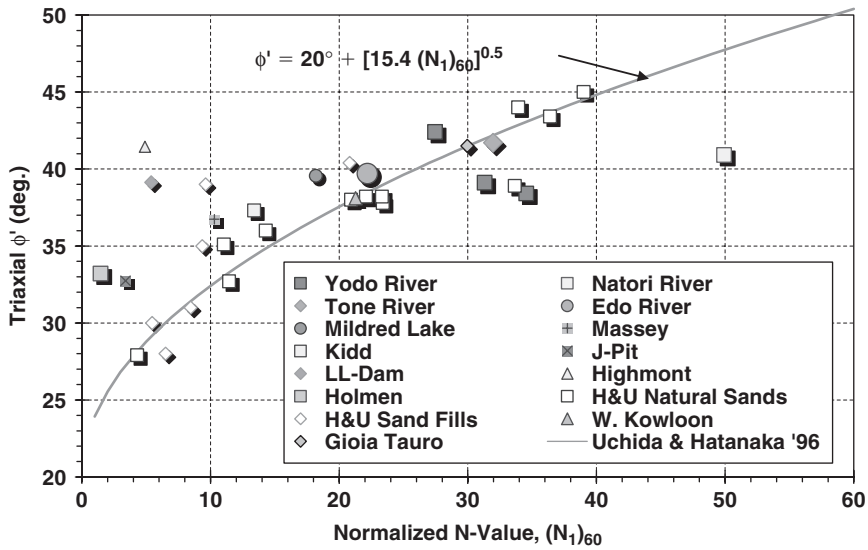


Figure 43. Effective friction angle of undisturbed sands vs. normalized SPT resistance in comparison with Hatanaka & Uchida (1996) relationship.

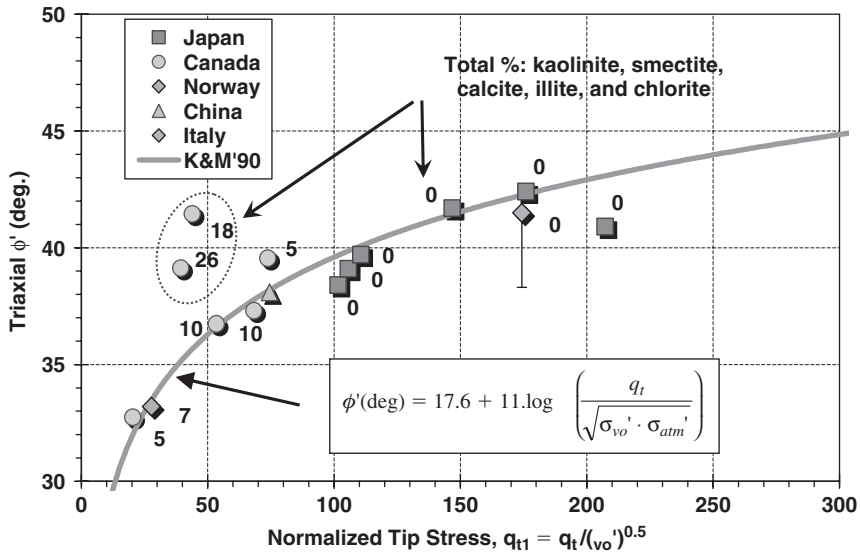


Figure 44. Effective triaxial friction angle of undisturbed sands vs. normalized cone tip stress.

unusual mineralogy, as compared with the other eleven sands. Also, the J-pit site was selected for the full-scale attempt at liquefaction for the CANLEX experiments, yet did not liquefy (Robertson, et al. 2000a,b).

3.7 Stress-strain-strength response of sands

The stress-strain-strength response of sands can be handled similarly to the aforementioned approach, using a variety of nonlinear expressions or algorithms to represent modulus reduction ($G/G_{\max}$), or alternatively using constitutive soil models. In the generalized modified hyperbola version by Fahey &

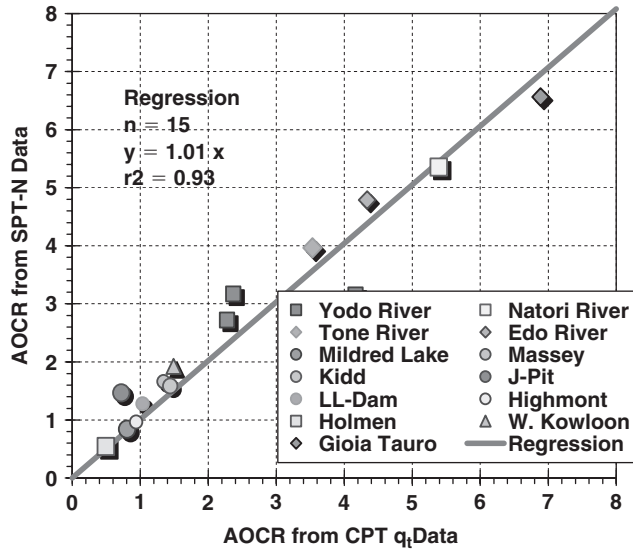


Figure 45. Apparent OCRs of undisturbed sands from SPT and CPT inferences.

Carter (1993), two parameters (f and g) are used to obtain the secant modulus reduction factors (Fahey, 1998) by one of the following:

$$G/G_{\max} = 1 - f(\tau/\tau_{\max})^g \quad (33a)$$

$$E/E_{\max} = 1 - f(q/q_{\max})^g \quad (33b)$$

For the above, no changes in ν' are made during loading, however, could be implemented as needed (e.g., Fahey & Carter, 1993). Poisson's ratio data on 6 sands presented by Lehane & Cosgrove (2000) show relatively constant ν' during loading up to about 0.1% strains, afterwards indicating increases in ν' . The expressions of (33) have been used effectively to represent stress-strain-strength curves in simple & torsion shear and triaxial modes for different soil types, including: clays (e.g., Elhakim & Mayne, 2003; Mayne et al. 2003), silts (Mayne et al. 1999), clean sands (Fahey 1998; Lehane & Fahey, 2002), and silty sands (Lee, et al. 2004). For uncemented soils that are not highly structured, adoptive values of $f = 1$ and $g = 0.3 \pm 0.1$ have been suggested for initial guestimates (Mayne 2005).

The drained strength of sands depends on the particular mode of loading application. For direct simple shear (DSS), the shear strength may be stated simply by: $\tau_{\max} = \sigma'_{vo} \cdot \tan \phi'$; while for isotropic triaxial drained compression tests (CIDC), the peak deviator stress is obtained from: $q_{\max} = (\sigma_1 - \sigma_3)_{\max} = 2 \sigma'_o \cdot \sin \phi' / (1 - \sin \phi')$. For torsional shear tests (TS) and anisotropically-consolidated triaxial tests (CADC), consideration of the initial K_0 state can be made by stress path analyses. For CADC, this gives: $q_{\max} = (\sigma_1 - \sigma_3)_{\max} = 2 K_0 \sigma'_{vo} \cdot [1/(1/\sin \phi' - 1)]$.

The use of the modified hyperbola for representing stress-strain-strength curves of sand is illustrated in Figure 46 for Natori River sand reported by Mimura (2003). Here, $f = 1$ and an exponent value $g = 0.3$ to 0.4 gives an overall good fitting.

While the aforementioned fitting technique was shown to be simple and reasonable for a given sand, it cannot effectively address complex stress paths and behavior, because of its empirical basis. The use of a constitutive soil model would therefore be beneficial since it may be capable of handling additional facets of soil behavior, including: drained and undrained loading, variable stress paths, volumetric strain predictions & dilatancy, cyclic response, and post-peak softening. Summaries of some available models are given by Duncan (1994) and Lade (2005).

Some constitutive models of interest include the rather versatile *NorSand* (Jefferies, 1993; Jefferies & Shuttle, 2005) which requires only 8 soil parameters and has been calibrated with several sands and clays. Also, the more sophisticated *MIT-S-1 model* (Pestana & Whittle, 1999) can produce a variety of stress paths, modes, and varied responses for both sands and clays. Consequently, future research should be directed towards the calibration of laboratory and in-situ tests within the framework of constitutive soil

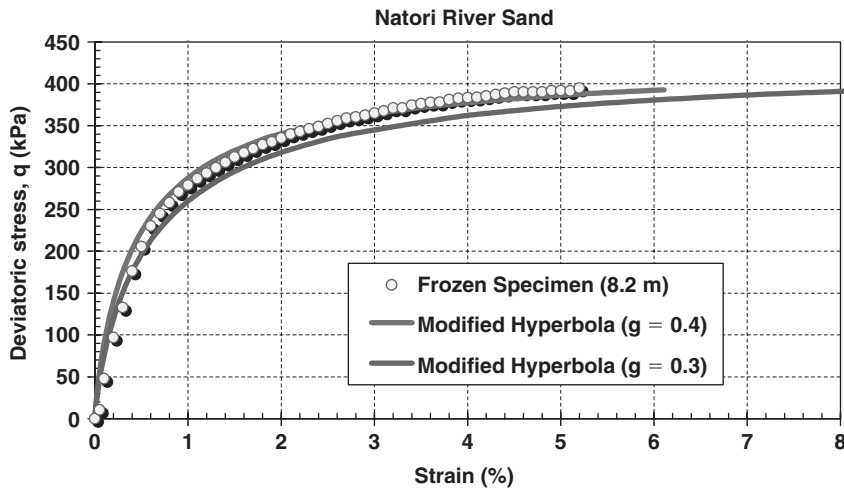


Figure 46. Triaxial stress-strain curve from undisturbed sand represented by modified hyperbola fitting (data from Mimura 2003).

models as these will allow versatility. The initiation of this database on undisturbed sands will help begin the important calibration of parameters as needed for these constitutive soil models.

4 IN-SITU TESTING OF INTERMEDIATE AND NONTEXTBOOK GEOMATERIALS

4.1 Other soil types

In nature, there are enumerable types of soils with varying components, packing arrangements, and origins. Yet, in the “ivory tower” of academe, it is usual to group soil behavior into two broad categories (as has been covered herein): *sands* (drained behavior) and *clays* (undrained behavior). Thus, anything outside of the “hourglass sands” and “vanilla clay” categories could be termed *nontextbook geomaterials*.

Within the CSSM context, all soils (clays, silts, sands, gravels) can exhibit drained to semi-drained to partially-undrained to fully-undrained conditions, given the particular circumstances. Mixed soils with varying percent fines, weathering, high structure, fabric, unusual mineralogical components, and particles have been encountered in geotechnical practice and do not fit within the empirical domain of existing correlations and procedures of analysis. As such, a consensus version of CSSM framework for unsaturated structured soils based in micromechanics would be desired to account for many additional facets of soil behavior (e.g. Cho & Santamarina, 2001; Leroueil & Hight, 2003) and is really beyond the scope & capabilities of this paper.

A number of recent papers have addressed the concerns and significance of in-situ test interpretation in nontextbook geomaterials. Such geomaterials are widespread and diverse, including: calcareous soils, carbonate sands, partially-saturated soils, diatomaceous clays, weathered residuum, lateritic soils, loess, mine tailings, and gaseous sediments. For these issues, the reader is directed towards the efforts reported in such references as Lunne et al. (1995; 1997), Hight & Leroueil (2003), Schnaid et al. (2004), and Schnaid (2005), as well as the many individual papers found in the 2003 and 2006 Singapore Workshop proceedings.

4.2 Global correlations

Of interest to the post-processing of in-situ data is the development of generalized correlations, as these may find use in the assessment of geotechnical parameters for all types of “well-behaved” soils. Soil behavioral charts have been produced that utilize in-situ readings and indirectly assess soil type. For instance, the DMT readings p_0 and p_1 give the material index ID that determines type of soil. Two readings from CPT (q_t and f_s) can be used to give approximate soil types in low sensitivity and noncemented geomaterials (e.g. Schmertmann 1978), else in CPTU, two readings could focus on q_t and B_q (Senneset

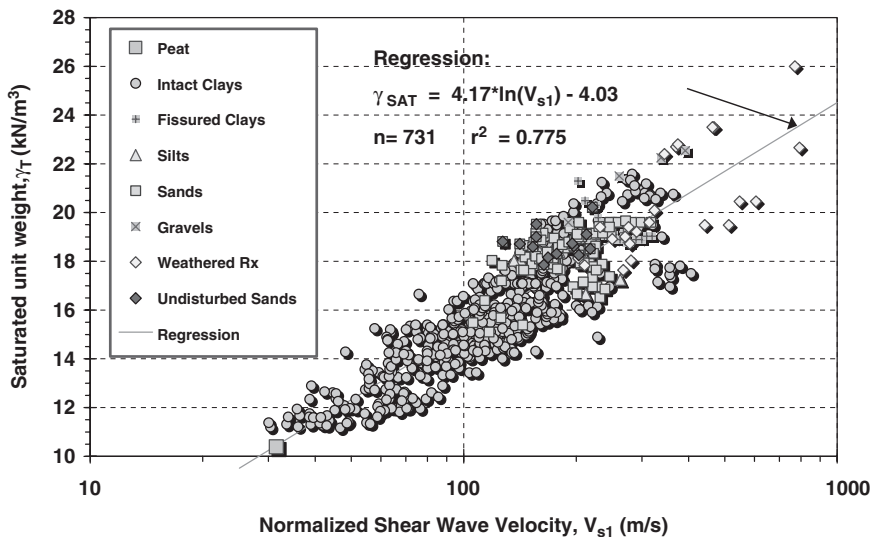


Figure 47. Saturated unit weight evaluation from stress-normalized V_{s1} in non-cemented soils.

et al. 1988). In fact, all three readings from CPTU (q_t , f_s , and Δu) can be used for soil behavioral identification (e.g., Robertson 1990).

These comparison of intra-measurements can be utilized to find “unusual” or anomalous soils that would classify as problematic geomaterials. For instance, Fahey (1998) and Schnaid et al. (2004) plot the ratio G_0/q_t which decreases with q_{t1} for unaged quartz and siliceous sandy soils. This baseline is then used to distinguish cemented or structured soils, as these tend to fall above this curve. Thus a cross-comparison of multiple readings taken during the same sounding helps to identify the soil type. With the seismic piezocone, four parameters are obtained (Q , F , B_q , G_0/q_t) to allow a better means to notice or define prospective nontextbook soils.

From a global database on soils ($n = 731$), the saturated unit weight (γ_{sat}) of “well-behaved” soils can be guesstimated from the stress-normalized shear wave velocity (V_{s1}) within about $\pm 1 \text{ kN/m}^3$, as suggested by Figure 46. The figure also includes the 14 undisturbed sands from Tables 2 to 4. Unit weights from 10 to 26 kN/m^3 are associated with normalized shear wave velocities from 30 to 800 m/s. As V_{s1} is dependent upon the current effective overburden stress, the procedure should be used in a depth-stepwise fashion, starting from the ground surface and proceeding downward. It can be expected that cemented soils and carbonate sands would lie somewhat to the right of the mean relationships shown, as the bonding would promote a more open porous structure (and low unit weight), yet a fast velocity in the matrix.

For dry soils above the water table and no capillarity effects, a similar relationship can be derived from lab resonant column test data. Figure 47 shows the reprocessed RCT data from four series of reconstituted quartz sands tests (Richart et al. 1970) in terms of dry unit weight vs. V_{s1} . Dry unit weights from 14 to 20 kN/m^3 are associated with normalized V_{s1} from 220 to 280 m/s.

The real difficulty with this indirect approach comes with partially-saturated soils in the field. Here, use of measured shear wave velocities in the vadose zone above the groundwater table is complicated because of capillarity effects. As the soil is desaturated, the shear wave velocity increases by two-fold to five-fold, the extent depending upon the grain size distribution of the soil (Cho & Santamarina, 2001). Thus, a measure of the degree of saturation may be necessary. Towards this purpose, Jamiolkowski (2001) has suggested that in-situ compression wave measurements (V_p) can aid in evaluating the degree of saturation.

With many well-documented geotechnical experimental test sites now established (such as those cases reported in the 2003 and 2006 Singapore Workshops), it is now feasible to seek comparisons across diverse soil types and formations, looking for more global (and presumably, more reliable) correlations that can serve as baselines in cross-checking site-specific data and generalized interpretation of soil engineering parameters. The author has made some initial steps towards this purpose using compiled SCPTU data from a variety of clays, silts, and sands, including many reported in the first Singapore proceedings (Tan et al. 2003). In addition to intact clays, the selection considers fissured clays, organic clays, and silts.

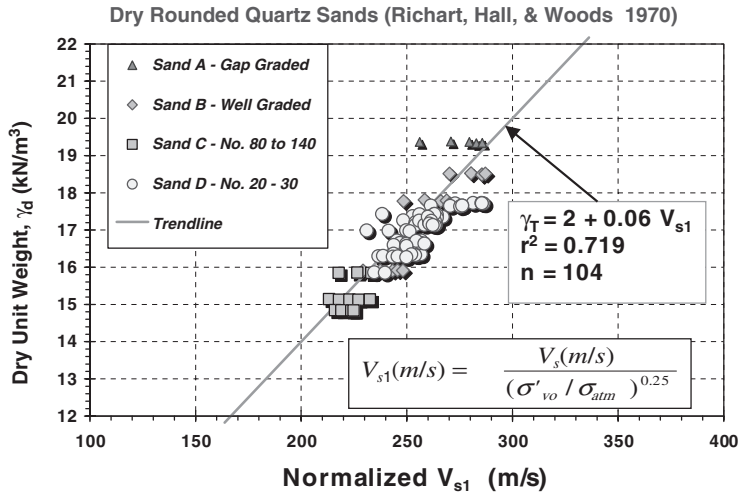


Figure 48. Trend of dry unit weight of sands in terms of stress-normalized shear wave velocity.

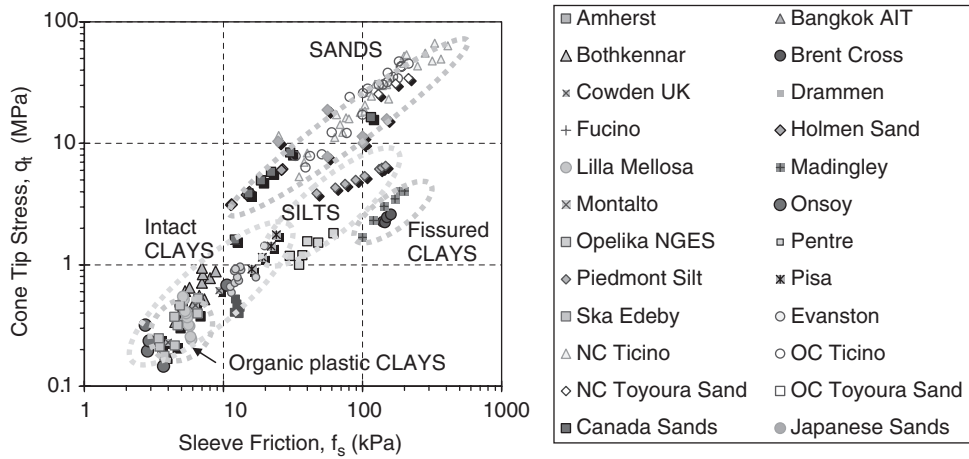


Figure 49. Combinative SCPTU database with q_t vs. f_s summary for varied clays, silts, and sands.

A collection of cross-comparisons from the CPTs from these sites is shown in terms of cone tip resistance vs. friction resistance in Figure 49. The undisturbed sands from Tables 2 to 4 are also included.

For foundation settlement analyses, a representative constrained modulus of the supporting soil medium is usually sought. The one-dimensional consolidation test (or oedometer) provides the constrained modulus ($D' = \Delta\sigma' / \Delta\epsilon$), although in some cases, the modulus values can be backcalculated from field performance data from embankment loadings or footings. In practice, it has been usual to further correlate the modulus D' to a penetration resistance, as in-situ penetration tests are the most common type employed in routine site investigations. From the global plotting of diverse geomaterials, Figure 50a shows that a relationship for “well-behaved” soils might take the form (Schmertmann 1978):

$$D' \approx \alpha'_C \cdot (q_t - \sigma_{v0}) \quad (34)$$

with a representative value of $\alpha'_C \approx 5$ for soft to firm “vanilla clays” and NC “hourglass sands”. However, for organic plastic clays of Sweden, a considerably lower $\alpha'_C \approx 1$ to 2 may be appropriate. For cemented Fucino clay, a value $\alpha'_C \approx 10$ to 20 may be assigned.

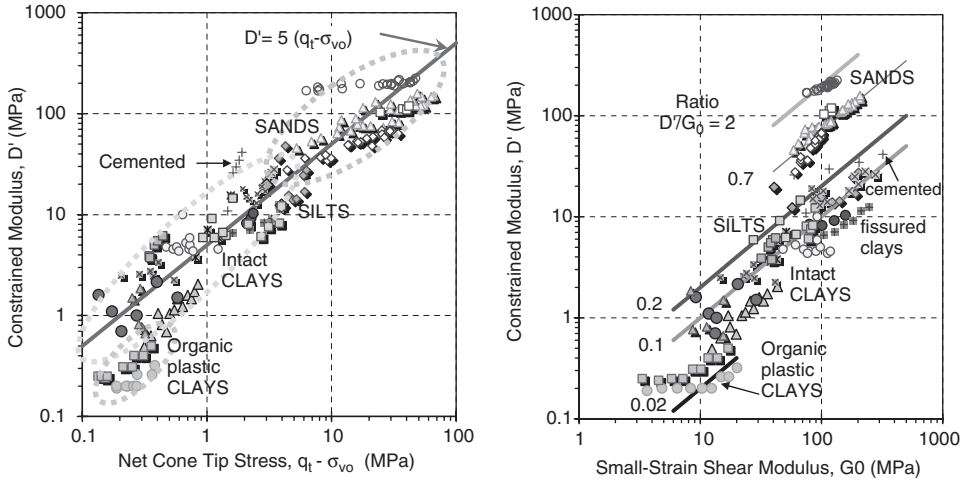


Figure 50. Combinative soil SCPTU database relationships between oedometric modulus (D') and (a) net cone tip resistance and (b) small-strain shear modulus.

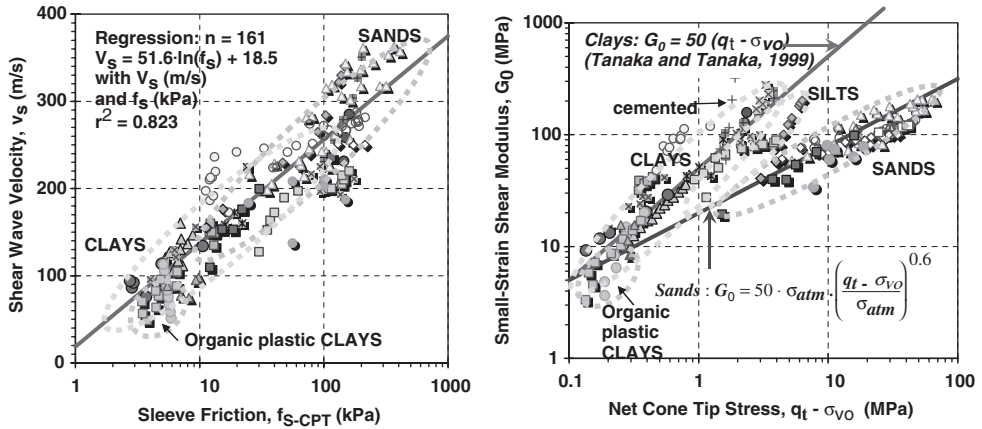


Figure 51. Observed SCPTU trends for non-calcareous soils: (a) shear wave vs. sleeve friction; and (b) small-strain stiffness and net cone tip resistance.

An alternate correlation can be sought between D' and small-strain shear modulus (G_0), as presented in Figure 50b. In this case, a similar adopted format could be:

$$D' \approx \alpha'_G \cdot G_0 \quad (35)$$

with assigned values of α'_G ranging from 0.02 for the organic plastic clays up to 2 for overconsolidated quartz sands. Additional studies with multiple regression, artificial neural networks, and numerical modeling may help guide the development of more universally-applied global relationships.

During the examination of various possible relationships in the above soil database, the results of two interesting trends became apparent (Figure 51). In Figure 51a, the measured V_s appeared related to the sleeve friction for uncemented soils. Figure 51b showed that the proposed relationship by Tanaka & Tanaka (1999) could be modified for soil type according to:

$$G_0 \approx 50 \sigma_{atm} \cdot [(q_t - \sigma_{vo}) / \sigma_{atm}]^{m^*} \quad (36)$$

where $m^* = 0.6$ for clean quartz sands, 0.8 for silts, and 1.0 for intact clays of low to medium sensitivity. Note the remarkable resemblance to the aforementioned (30).

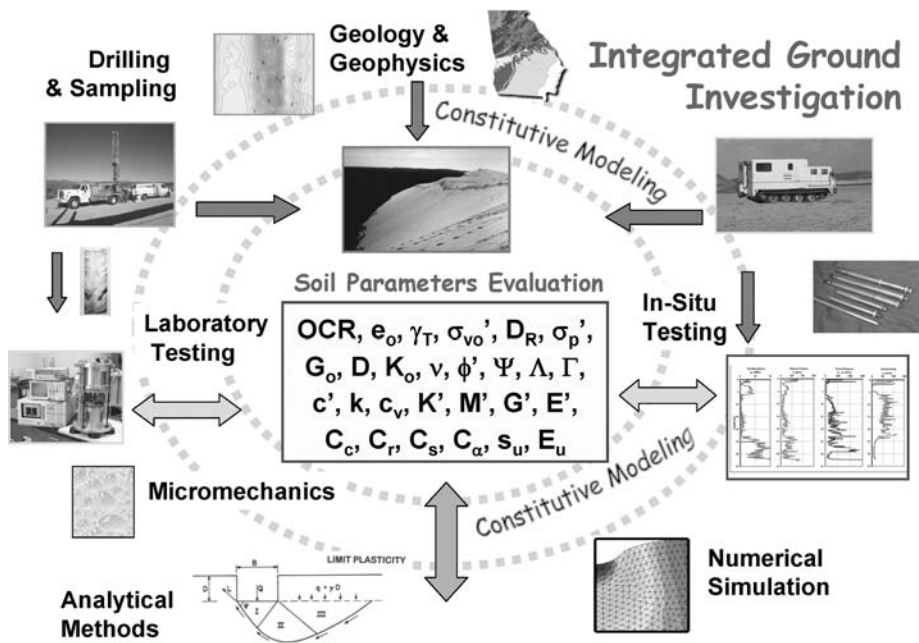


Figure 52. Concept for an integrated geotechnical approach to ground investigations.

5 CONCLUSIONS

The interpretation of in-situ tests in soils is currently a mix & match of theoretical, analytical, and empirical relationships that have developed separately and independently based on single-focused needs and research studies. Yet now, a sufficiently high number of well-documented clay and sand sites currently exists to allow a comprehensive calibration of a full suite of in-situ tests within a consistent framework. The results can be validated using high-quality laboratory test data and/or parameters backfigured from full-scale load tests. What is needed is a unified framework based on numerical modeling of all major test types using a rigorous constitutive soil model (likely within the context of critical state soil mechanics). This should be an integrated approach using geophysics, drilling & sampling, laboratory and in-situ testing (Fig. 52).

In the interim, this paper shows that stress history (i.e., OCR and OCD) can play a significant role in unifying the approach to in-situ test interpretations in soils. As a suggested first step, the utilization of a simplified cavity expansion – CSSM approach to fitting CPT, CPT_u, and DMT data to clay sites is presented for six sites that range from soft NC to stiff OC states. For clean sands, an empirical approach using SPT, CPT, and DHT V_s data is applied to several well-documented sands and a special undisturbed (frozen) sand database is presented to cross-check relationships developed from large scale calibration chamber test series. Procedures for nontextbook geomaterials are also in dire need of a common methodology that can address clays and sands, as well as intermediate soil types and unusual to difficult soils. Additional global relationships for “well-behaved” and “normal” soils are sought in order to identify problematic and “ill-behaved” geomaterials. In-situ tests which obtain multiple-measurements (i.e., SCPT_u and SDMT_a) offer the most promise as intra-comparative cross-checks amongst the readings can be used to help discern anomalous soil behavior, as well as provide a suite of engineering parameters for “normal” soils, including stress-strain-strength-flow characteristics. Consequently, it is quite evident that considerable work remains in the development of a rational and validated approach to geotechnical site characterization by in-situ tests.

ACKNOWLEDGMENTS

The author appreciates the financial support provided currently by the Georgia Dept. of Transportation (GDOT), as well as prior support given by the National Science Foundation (NSF) and Fugro Engineers

BV. Any conclusions or findings reported herein do not necessarily reflect the procedures or guidelines of these groups.

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Soil variability analysis for geotechnical practice

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ABSTRACT: The heterogeneity of natural soils is well known in geotechnical practice. However, the importance of quantifying the resulting variability in geotechnical characterisation and design parameters is not adequately recognised. Variability should not be approached suspiciously. Rather, it should be accepted as a positive contribution to geotechnical design as its consistent modelling and utilisation lead, with limited additional computations and conceptual effort on the part of the engineer, to more rational and economic design. The paper presents a structured – though necessarily partial – review of approaches and methodologies for the quantification of soil variability, as well as selected examples of its utilisation in reliability-based geotechnical design.

1 INTRODUCTION

Soils are naturally variable because of the way they are formed and the continuous processes of the environment that alter them. After deposition or initial formation, they are modified continuously by external stresses, weathering, chemical reactions, introduction of new substances and, in some cases, human intervention (e.g., soil improvement, excavation, filling).

In a geotechnical perspective, three levels of soil heterogeneity could be defined. *Stratigraphic heterogeneity* is the result of large-scale geologic and geomorphological processes. This heterogeneity is usually addressed at site-scale; stratigraphies may be extremely complex and heterogeneous. This level of heterogeneity is commonly addressed by the geotechnical engineer in the context of site characterisation, in which the geotechnical engineer expects the existence of soil units which are strongly heterogeneous from a compositional or mechanical point of view.

Lithological heterogeneity can be manifested, for instance, in the form of thin soft/stiff layers embedded in a stiffer/softer media or the inclusion of pockets of different lithology within a relatively uniform soil mass.

Inherent soil variability is the variation of properties from one spatial location to another inside a soil mass which could be regarded as being significantly homogeneous for geotechnical purposes. At this level, it is *necessary* to assign *quantitative values* to parameters of interest; such values should be *representative* of the parameters in the soil unit.

The conceptual relevance of recognising soil heterogeneity does not *decrease* on a smaller scale. The roles of the three types of heterogeneity in the engineering process are significantly different. Large-scale stratigraphic heterogeneity is important in the characterisation phase and for preliminary design decisions; medium-scale lithological heterogeneity may affect the choice of the engineering model to obtain a design value for a parameter of interest; a proper analysis of inherent variability can become a powerful tool in the hands of the engineer to achieve more rational and economic design.

The term “inherent variability” used in this paper only applies to scales measurable by common geotechnical testing. There is currently a knowledge gap between our physical understanding of microstructure and how statistical variations at micro-scale impact engineering-scale systems. For instance, the modulus degradation curve provides us with a glimpse of the underlying complexities in natural soils. Comparable variations in strains do not translate to comparable variations in modulus

because of nonlinear behaviour. Hence, variations in modulus deduced from large-strain in-situ tests cannot be extrapolated to variations at smaller scales. Nevertheless, it is well known that ground settlement behind deep excavations depend on small-strain stiffness and variations in ground settlement would thus depend on variations in small-strain stiffness. These variations potentially can be characterized by shear wave velocity and comparable small-strain non-intrusive measurements (e.g., seismic CPT) that are gradually adopted in practice.

The engineer processes available data to obtain parameters which are useful for characterisation and design. Far from being a merely technical issue, the choice of the approach by which a geotechnical engineer creates a model of physical reality as he is given to see: *from data which are never completely precise and accurate*, reflects a philosophical choice which is, nonetheless, heavily dependent on – and constrained by – regulatory, economic and technological factors. Vick (2002) addressed the complex universe of decision-making in geotechnical reasoning and its many nuances.

The choice of representative parameters, whichever the geotechnical approach, recognises the inherent variability of soils. However, the *level of explicitness* with which variability is included in the assignment procedure, depends upon the selected approach. In *deterministic approaches*, inherent variability does not appear as explicitly as in *uncertainty-based approaches*.

In principle, no category of approaches is preferable over another. The quality of geotechnical characterisation or design is not conceptually bound to the level of explicitness of soil variability modelling. Deterministic methods lie at the basis of virtually every technological science, and geotechnical engineering is no exception. However, collective experience (both from practice and research) suggests that it may be time for a shift to an uncertainty-based perspective which may be, on the whole more *convenient* in terms of *safety*, *performance* and *economy*.

The main advantage of uncertainty-based methods over deterministic methods in terms of *safety* lies in the fact that, due to the explicit inclusion of uncertainty in inputs and the explicit declaration of the level of uncertainty in the outputs, the former are able to provide more *complete* and *realistic* information regarding the level of safety of design: soils *are* variable, whether such variability is recognised in design or not! Addressing uncertainty does not increase the level of safety, but allows a more rational design as the engineer can consciously calibrate his decisions on a desired or required *performance level* of a structure. Being able to select and communicate the level of performance and reduce undesired conservatism, in turn, is beneficial in the economic sense. Hence, geotechnical practitioners should envisage soil variability as a *positive* factor for design.

Perhaps the main trade-off for the positive aspects of uncertainty-based approaches, which has hindered its dissemination among geotechnical practitioners, is the necessity to rely on somewhat more complex estimation approaches involving. It is necessary to refer to mathematical techniques which address uncertainty. While ever-increasing computational capabilities and new dedicated software gradually remove some of the barriers encountered by research and practitioners in the past (e.g. the computational expense of implementing Monte Carlo simulation), variability analysis requires at least some degree of comprehension on the side of the engineer who performs it or even assesses its results. Engineering decision and judgment should never be completely replaced by automated computational procedures, however powerful these may be.

1.1 *Variability vs. uncertainty*

In the engineering literature – and geotechnical engineering is no exception – the terms *variability* and *uncertainty* are often employed interchangeably. While it is true that the two terms refer to concepts which are significantly related, and that the frequently occurring terminological superposition is unlikely to result, in practice, in unfavourable consequences, it is opined here that a clarification would positively contribute to a progressive, virtuous increase in the terminological rigour in the geotechnical literature.

In the technical sense, *variability* can be defined as an observable manifestation of heterogeneity of one or more physical parameters and/or processes. In principle, a variable property could be described, for instance, if a sufficient number of measurements were available and if the quality of the measurements themselves was sufficient to ensure confident evaluation of the observations. Hence, the observation of variability implicitly provides a more or less detailed assessment of the level of knowledge on a phenomenon of interest and of the capability to measure and model the phenomenon itself.

Uncertainty reflects the decision (or necessity) to recognise and address the observed variability in one or more soil properties of interest. In many cases it is not possible (given the inherent nature of a physical process and the available level of knowledge and/or technology related to the process itself) to model the variability in a completely satisfactory way. On the other hand, anticipating a fundamental concept of soil variability modelling, it may be deemed *advantageous* to *assume* that a phenomenon

is indeterminate (uncertain) to a certain extent in cases where a detailed description of variability is expected to be uneconomic or redundant.

1.2 Applications of soil variability modelling

The main practical applications of soil variability modelling are: (a) geostatistics, focusing on interpolation of available data values to estimate other unavailable data values at the same location; and (b) geotechnical engineering, that focuses on characterisation for reliability/risk assessment. While the theoretical aspects underlying such applications are not the topic of the paper, it is deemed useful to provide a concise description of each.

1.2.1 Geostatistics

A well known problem in geotechnical site characterisation is the limited amount of available data. It is frequently necessary to estimate soil properties at specific locations where they have not been observed. To do so, it is necessary to interpolate the available data while accounting for the spatial correlation of soil properties of interest. Geostatistics makes use of a specific modelling approach to such spatial interpolation known as *geostatistical kriging*.

Kriging is essentially a weighted, moving average interpolation procedure that minimises the estimated variance of the interpolated value with the weighted average of its neighbours. The input information required for kriging includes: available data values and their spatial measurement locations; information regarding the spatial correlation structure of the soil property of interest; and the spatial locations of target points, where estimates of the soil property are desired. The weighting factors and the variance are computed using the information regarding the spatial correlation structure of the available data. Since spatial correlation is related to distance, the weights depend on the spatial location of the points of interest for estimation.

The works of G. Matheron, D.G. Krige, and F.P. Agterberg in founding this area are notable. It should be acknowledged that parallel developments also took place in meteorology (L.S. Gandin) and forestry (B. Matérn). Finally, geostatistics is fundamentally based on the theory of random processes/fields developed by mathematicians A.Y. Khinchin, A.N. Kolmogorov, P. Lévy, N. Wiener, and A.M. Yaglom, among others. Many forms of kriging have been developed since; the reader is referred to, for instance, Journel & Huijbregts (1978), Davis (1986) and Carr (1995).

Nadim (1988) and Lacasse & Nadim (1996) provided applications of geostatistical kriging. Figure 1a (Lacasse & Nadim 1996) shows the locations of cone penetration test soundings in the neighbourhood of a circular-shaped shallow foundation which was to be designed; Figure 1b reports the superimposed profiles of cone resistance in the soundings. It was of interest to estimate the values of cone resistance in other spatial locations under the design location of the foundation itself. Figure 2 presents the contours of cone penetration resistance at each metre as obtained by kriging. The 3D graphic representation provides improved insight into the possible spatial variation of cone resistance and the most likely values beneath the foundation. The results of the analysis enabled designers to determine more reliably the position of a clay layer and to use higher shear strength in design.

1.2.2 Reliability-based geotechnical design

The practical end point of characterising uncertainties in the design input parameters (geotechnical, geo-hydrological, geometrical, and possibly thermal) is to evaluate their impact on the performance of a design. Reliability analysis focuses on the most important aspect of performance, namely the probability of failure (where “failure” is a generic term for non-performance). This probability of failure clearly depends on both parametric and model uncertainties. The latter are not covered in this paper. The probability of failure is a more consistent and complete measure of safety because it is invariant to all mechanically equivalent definitions of safety and it incorporates additional uncertainty information. There is a prevalent misconception that reliability-based design is “new”. All experienced engineers would conduct parametric studies when confidence in the choice of deterministic input values is lacking. Reliability analysis merely allows the engineer to carry out a much broader range of parametric studies without actually performing thousands of design checks with different inputs one at a time. This sounds suspiciously like a “free lunch”, but exceedingly clever probabilistic techniques do exist to compute the probability of failure efficiently. The chief drawback is that these techniques are difficult to understand for the non-specialist, even though they may not necessarily be difficult to implement computationally. There is no consensus within the geotechnical community if a more consistent and complete measure of safety is worth the additional efforts or if the significantly simpler but inconsistent global factor of safety should be dropped. Regulatory pressure appears to be pushing towards reliability-based design for

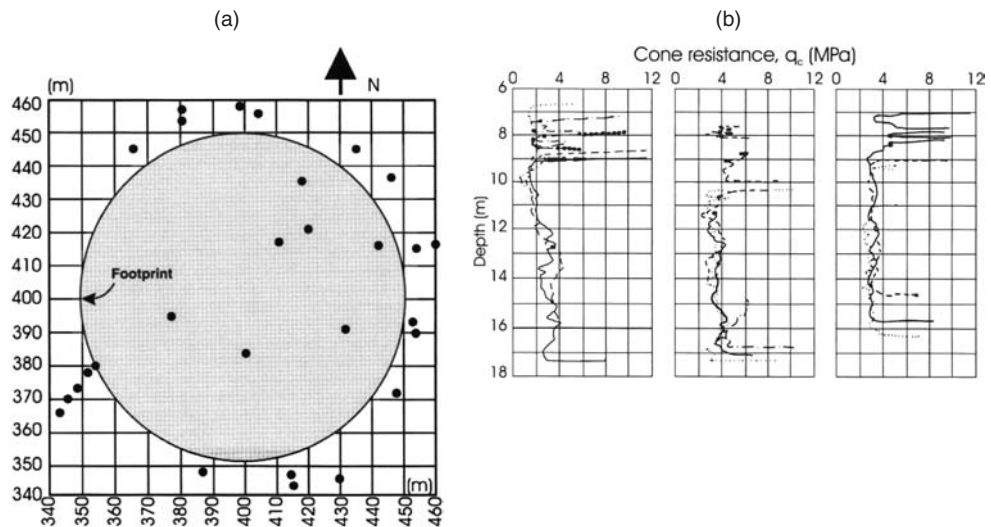


Figure 1. (a) Locations of cone penetration test soundings; (b) Superimposed profiles of cone resistance (Lacasse & Nadim 1996).

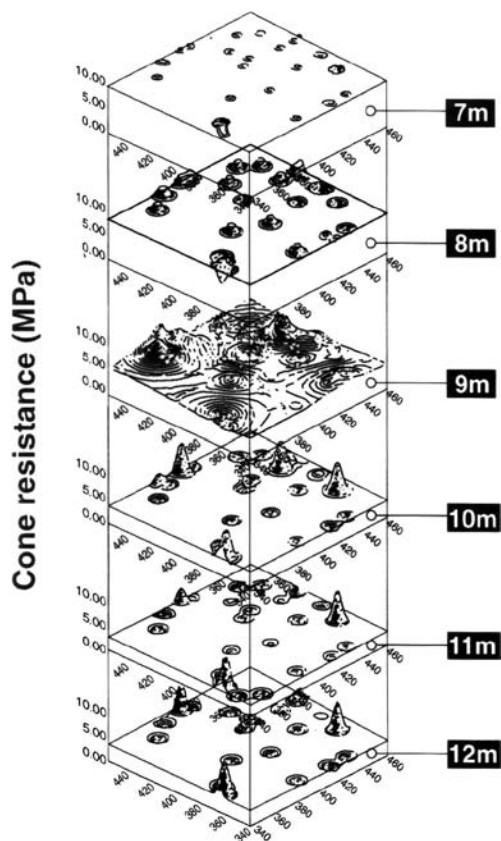


Figure 2. Contours of cone penetration resistance at each metre as obtained by geostatistical kriging (Lacasse & Nadim 1996).

Table 1. Factors determining the character of geomaterials (Hight & Leroueil 2003).

Formative history
Sedimentary
Depositional environment (alluvial, marine, Lacustrine, glacial, estuarine)
Post-depositional processes (cementing, bioturbation, leaching, desiccation, ageing, tectonics, lithification, chemical alteration, weathering)
Residual
Form and intensity of weathering
Parent rock
Age
Composition
Complete grading
Grain/aggregate size, shape, texture and strength
Silt fraction – shape – plasticity
Clay fraction – mineralogy – form (clay minerals, rock flour, biogenic debris)
Stability
Organic content and form
Pore water chemistry (salinity, sulphates, pH, etc.)
Fabric
Macrofabric
Interbedding, laminations, discontinuities (faults, joints, fissures, open cracks)
Microfabric
Orientation, density variations, bioturbation features, pore size distributions, local void ratio
Microstructure
Cementation – form, distribution, strength
Ageing effects (including creep)
Recent stress/strain history
Unloading/reloading
Groundwater level fluctuations
Wave loading
Ground movements
Sampling
State
Water content, void ratio, degree of saturation
Density, relative density
In-situ stresses (including pore pressure) and location relative to limit state curve
Current strain rate and time effects
Drainage conditions
Drained, partially drained, undrained (permeability, drainage path lengths, rate of stress [or strain] change, pore pressure gradients)
Stress/strain path imposed by construction and subsequent loading
Rate of stress or strain change
Disturbance/destructuring
Temperature

non-technical reasons. The literature on reliability analysis and reliability-based design is voluminous. The reader is referred to Phoon et al. (2003a, 2003b) for a summary and more detailed reference listings.

1.3 *Factors inducing soil variability*

The spatial variation of a soil property is believed to be more related to formation processes rather than to the chemical and mechanical details of the soil particles themselves. This has been asserted both for natural soils and for man-made soils (e.g. Fenton 1999a). Larsson et al. (2005) specifically addressed variability in man-made soils. Table 1 (Hight & Leroueil 2003) provides a list of the main factors contributing to the definition of the character of geomaterials.

Spatial variability may also be influenced by artificial disturbances such as construction or induced groundwater level variations. Breyse et al. (2005) provided a set of references addressing the factors which are relevant to the definition of the spatial correlation structure of a soil property.

The present paper focuses on the quantitative modelling of inherent soil variability; hence, the physical phenomena contributing to variability will be addressed, when pertinent, in the light of their effect on

quantitative estimation. The interested reader is referred to Hight & Leroueil (2003) and Locat et al. (2003) for further insights into the physical causes of variability.

1.4 Geotechnical uncertainty

The total uncertainty in a measured or design geotechnical parameter may be addressed quantitatively by *uncertainty models*. Several models have been proposed in the literature (e.g. Phoon & Kulhawy 1999b). Uncertainty models identify inherent soil variability, measurement error, statistical estimation error and transformation error as the primary sources of uncertainty. Inherent variability gives rise to *aleatory uncertainty*, while measurement error, statistical estimation error and model error contribute to *epistemic uncertainty* (e.g. Lacasse & Nadim 1996; Phoon & Kulhawy 1999a). The present paper focuses on inherent soil variability or, in other words, on the quantification and effects of aleatory uncertainty on geotechnical characterisation and design.

It is deemed useful to describe the components of epistemic uncertainty in extreme synthesis. *Measurement uncertainty* is due to equipment, procedural-operator, and random testing effects. Equipment effects result from inaccuracies in the measuring devices and variations in equipment geometries and systems employed for routine testing. Procedural/operator effects originate from limitations in test standards and how they are followed. Random measurement error refers to the remaining scatter in the test results that is not assignable to specific testing parameters and is not caused by inherent soil variability. It is usually possible to identify a *measurement bias* (i.e. a consistent overestimation or underestimation of the real value of the object parameter) and a model dispersion. Orchant et al. (1988) discussed the factors contributing to measurement uncertainty for geotechnical laboratory and in-situ testing. Kulhawy & Trautmann (1996) and Phoon & Kulhawy (1999a) provided tabulated values of measurement uncertainties for laboratory and in-situ tests.

Transformation uncertainty results from the approximations which are inevitably present in empirical, semi-empirical or theoretical models commonly used in geotechnical engineering to relate measured quantities to design parameters. This transformation or model error may include a *model bias* and a *model dispersion*. Such measures of uncertainty are of great importance in uncertainty-based design approaches such as reliability-based design. Model statistics can only be evaluated by: (a) realistically large-scale prototype tests; (b) a sufficiently large and representative database; and (c) reasonably high-quality testing in which extraneous uncertainties are well controlled. Generally, available testing data are insufficient in quantity and quality to perform robust statistical assessment of model error in most geotechnical calculation models. The reader is referred to Ang & Tang (1975), Baecher & Christian (2003) and Phoon & Kulhawy (2005) for a more detailed insight into model uncertainty.

Data sets are always finite in size. As will be discussed in §2, statistical inferences from any finite set of data are affected by *statistical estimation uncertainty*. The magnitude of this uncertainty may be relevant in the case of small data sets.

In quantitative uncertainty models, the various components of uncertainty are assumed to be uncorrelated. However, this is only approximately true. The quantification of spatial variability requires data from measurements of soil properties of interest. Different geotechnical measurement methods, whether performed in the laboratory or in-situ, will generally induce different failure modes in a volume of soil. This usually results in different values of the same measured property. Measured data are consequently related to test-specific failure modes. The type and magnitude of spatial variability cannot thus be regarded as being inherent properties of a soil volume, but are related to the type of measurement technique.

The hypothesis of uncorrelated uncertainty sources, though approximate, is especially important as it justifies separate treatment of inherent soil variability and the remaining components of total variability.

In terms of uncertainty, the aleatory uncertainty related to the inherent variability of soils is significantly different from the epistemic uncertainty: while the latter can be reduced by improving the quality of data and models (i.e. decreasing measurement and transformation uncertainty, respectively) and collecting more data (i.e. reducing statistical estimation error), aleatory uncertainty cannot be reduced, and can even increase as a consequence of an increase in the amount of data.

Results of past research (e.g. Orchant et al. 1988) indicate that the determination of aleatory uncertainty is significantly more reliable than the direct quantification of epistemic uncertainty.

1.5 Objectives and structure of paper

Inherent soil variability is a vast and complex topic. Even the investigation of a facet of variability, namely its modelling in an uncertainty-based perspective, is excessively ambitious if comprehensiveness

is sought. Far from attempting to provide a thorough review of the available literature, this paper aims to:

- Define spatial variability in the context of other types of variability encountered in the geotechnical discipline;
- Communicate the importance and convenience of investigating spatial variability;
- Provide an applicative insight into statistical and probabilistic modelling of geotechnical data;
- Illustrate a number of commonly used techniques for the quantitative description of spatial variation of soil properties;
- Report techniques which allow modelling of important physical phenomena in quantifying the effects of inherent variability on engineering design;
- Explain the existence of different levels of analysis of soil variability, identifying advantages and disadvantages in the context of design;
- Provide example applications of reliability-based techniques making use of the results of variability analyses;
- Provide updated tables of literature values of parameters of variability at the different levels of analysis while highlighting the limitations in the domain of applicability of literature data.

The rationale is to define a “best-practice” framework for the quantification and the communication of inherent soil variability parameters. The specific constitutive phases of a best-practice procedure are not static in time (research is ever ongoing) and are not univocally defined, as the choice of the most appropriate technique to address a specific problem is bound to the characteristics of available data.

Quantitative estimation of variability must forcedly rely on techniques which address uncertainty. Here, methods from statistics and probability theory as well as time series analysis are referred to. As this paper aims to stimulate the use of such methodologies by practicing engineers, the attempt is made to reduce the mathematical formalism as much as possible, ensuring a format suitable for application purposes.

The definition of common and consistent terminology and mathematical notation is a difficult task, given the formal and substantial heterogeneity in the many disciplines which contribute to soil variability investigations. Nonetheless, no effort was spared to achieve this goal and improve the flow and readability of the paper.

Data tables containing literature values of relevant output parameters of inherent variability are provided. However, such data should only be viewed as an example, and not used for reference of application purposes, unless it is verified that the *target site* in which they are being applied is sufficiently similar to the *source site* from which it was obtained. As important information regarding the in-situ characteristics of the source site and relevant details of the techniques are seldom specified even in research efforts, it is generally incorrect to use single, specific literature numerical values uncritically. It is more useful to view such data as a plausible range of values.

Investigation on soil variability can be performed at various levels of complexity, depending on the scope of the analysis, the quantity and quality of available data and the computing resources available.

At present, a “best-practice” soil variability investigation should address, for a parameter of interest, at least: (a) classical descriptive and inferential statistical analysis (e.g. estimation of mean, coefficient of variation and probability distribution); (b) spatial correlation structure describing the variation of the soil property from one point to another in space; (c) identification of the magnitude of spatial continuity, beyond which no or small correlation between soil data exists; and (d) spatial averaging and variance reduction, which help assess the reduction in the variance upon averaging over a volume of interest.

Phases (a)–(d) are listed in increasing order of complexity, and can be addressed using well established mathematical techniques from statistics and time series analysis. The phases are sequential in the sense that higher-level analyses require the results of lower-level analyses.

The trade-off for the simplicity of lower-level analyses lies in the limited generality of results as well as in the incomplete modelling of the behaviour of geotechnical systems. Reliability-based design can be achieved for different levels of complexity; however, higher levels of complexity allow consideration of additional effects and parameters which improve the quality of the reliability estimates.

The structure of the paper essentially reflects the ideal progression in complexity of a best-practice soil variability analysis. Section 2 briefly reviews descriptive and inferential statistical techniques for probabilistic modelling of soil properties in the second-moment sense. Section 3 offers a review of the most commonly adopted methods for the modelling of spatial variability of soil properties and for the identification of the spatial correlation structure. Section 4 addresses the parameters which are able to describe a spatially variable set of data concisely in the context of random field theory, as well as

introducing the important concepts of spatial averaging and variance reduction. Section 5 and Section 6 address in detail two fundamental aspects of soil variability analysis: respectively, the identification of physically homogeneous soil units and the verification of statistical independence of data sets.

In recent years, integrated approaches making use of Monte Carlo simulation, finite element analysis and the results of high-level soil variability investigations have been proposed. These approaches allow enhanced modelling of the real behaviour of geotechnical systems, in which the spatial heterogeneity of soil properties invariably plays an important role. While these studies are prevalently confined to a research perspective at present, they may provide very useful support to code-writers and practitioners in the future. An overview of applications of such techniques is provided in Section 7.

Section 8 offers some closing remarks.

Sections 2, 3, 4 and 7 are conceptually sequential, as they address soil variability modelling with increasing complexity, consistently with the aforementioned levels of analysis. A number of fundamental assumptions regarding the physical and statistical homogeneity of soils are common to all levels; these are addressed in Section 5 and 6, respectively.

The contents of Sections 2, 3 and 4 can be used both for geostatistical analyses and for reliability-based design. The practical implementation of the soil variability techniques presented herein is *not* the central topic of the present paper. A comparative review of reliability-based slope stability analyses from the geotechnical literature are provided at the end of Sections 2, 4 and 7. Section 7 also reports literature examples for other geotechnical system typologies such as soil-foundation systems, water-retaining earth structures, underground pillars and liquefaction-prone soil masses.

2 STATISTICAL AND PROBABILISTIC MODELLING OF SOIL VARIABILITY

Among available techniques for the investigation of uncertainty, the combined use of probability and statistics is the most frequently encountered in the geotechnical engineering literature. The reasons for the widespread reference to such approach are likely to be manifold. Probability and statistics rely on a very consistent bulk of literature; moreover, many concepts are intuitive to some degree.

2.1 *Probability and statistics for uncertainty analysis*

Probability and *statistics* form two distinct branches of mathematical knowledge. In practice, they are most often used synergistically and iteratively.

The formal mathematical aspects of probability theory and probability distributions are not addressed comprehensively herein. If a geotechnical perspective on probability theory is sought, the interested reader is referred, for instance, to Ang & Tang (1975), Smith (1986), Harr (1987), Rétháti (1988) or Baecher & Christian (2003).

Here, it is perhaps useful to distinguish, though in extreme synthesis, their respective contributions in an applicative perspective.

Statistical theory encompasses a broad range of topics. Among these, *descriptive statistics* deals with the representation of the variability in data in a conventional form and with the fitting of probability distribution functions to sample data, while the goal of *inferential statistics* is the modelling of patterns in the data accounting for randomness and uncertainty in order to draw general inferences about the variable parameter or process.

In other words, the descriptive perspective is used to describe as well as possible a particular data set (*source set*), with a view to interpolating within the data set. An inferential approach is required when it is of interest to investigate the value and variability of random properties of a set (*target set*) which can be expected to be in some way similar to the source set, but for which no or little quantitative information is available *a priori*. In the inferential perspective, probability theory provides the framework by which the outputs of statistical analysis can be processed consistently to provide a rational assessment of the propagation and the effects of uncertainty with reference to a specific problem.

Figure 3 provides a visualisation of the descriptive and inferential statistical approaches in the context of geotechnical engineering. In descriptive approaches, the source site (i.e. where measurements are available) coincides with the target site, and it is of interest to estimate properties at unmeasured locations. In an inferential approach, results of analysis performed on data from the source site are applied to a target site different from the source site, and where no measurements are available.

2.2 *Frequentist vs. degree of belief: the dual nature of probability*

Probability theory is useful in modelling the *observed* behaviour of a variable parameter if a set of measurements are available. However, an *assumed* behaviour can also be modelled. In the first case,

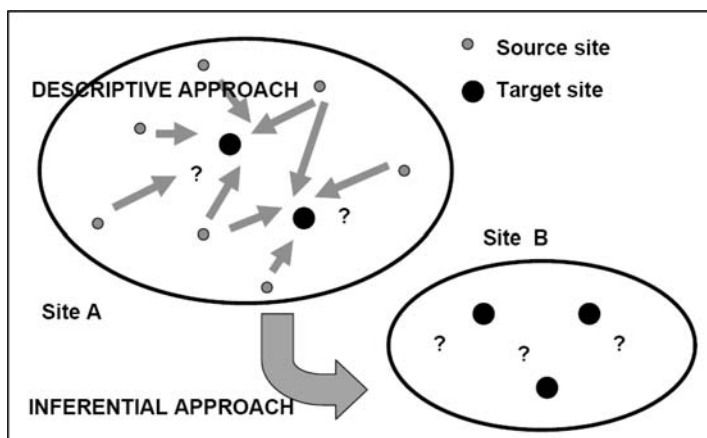


Figure 3. Descriptive and inferential statistical approaches in geotechnical engineering.

the *frequentist* nature of probability is referred to; in the second case, the *degree of belief* approach is pursued.

Vick (2002) and Baecher & Christian (2003) provide essays on the dual nature of probability. An important distinction between the two lies in the fact that estimates are essentially objective (i.e. obtained through repeatable numerical procedures) in the first case and mostly subjective (i.e. relying on expert knowledge and judgment) in the second.

Quantitative assessment of soil variability modelling requires use of descriptive and inferential statistics, as well as probabilistic modelling to process data from laboratory or in-situ measurements. Hence, the paper addresses probabilistic techniques in a frequentist perspective.

The phases leading to the probabilistic modelling of a random variable are shown schematically in Figure 4. The descriptive analysis includes the calculation of sample moments and (according to good practice) visual inspection of data and histograms. Inferential analysis includes the selection of a distribution type, the estimation of distribution parameters and goodness-of-fit testing of the resulting distribution to the source data. The dashed lines indicate that the results of the descriptive analysis *can* be used in the inferential analysis; however, inference could also be performed without prior statistical description.

Descriptive and inferential statistical approaches to probabilistic modelling are addressed hereinafter in an applicative perspective. The following treatment of the subject in no way intends to provide a comprehensive overview of the underlying statistical and probabilistic theoretical frameworks. The interested reader is referred to, for instance, the textbooks by Ang & Tang (1975) and Baecher & Christian (2003).

2.3 Descriptive analysis

Any quantitative geotechnical variability investigation must rely on sets (in statistical terms, *samples*) of measured data which are limited in size and quality. Hence, it is necessary to refer to *sample statistics*. Sample statistics are imperfect *estimators* of the “real” *population parameters*. Hence, they are never *completely* representative of the real distribution of the data, and are *biased* to some degree. Statements regarding the ability of estimators to approximate the true population *parameters* can be made on the basis of statistical theory.

The term *sample statistic* refers to any mathematical function of a data sample. For most engineering purposes, sample statistics are more useful than the comprehensive frequency distribution (as given by the histogram, for instance). An infinite number of sample statistics may be calculated from any given data set. For the practical purpose of inferential modelling, however, it is usually sufficient to calculate the first four statistical moments of a sample, i.e. the mean, variance, skewness and kurtosis. Higher moments are unreliable when estimated from most practical sample sizes.

The *sample mean*, i.e. the mean of a sample $\psi_1, \dots, \psi_n$ of a random variable Ψ is given by

$$m_{\psi} = \frac{1}{n} \sum_{i=1}^n \psi_i \quad (1)$$

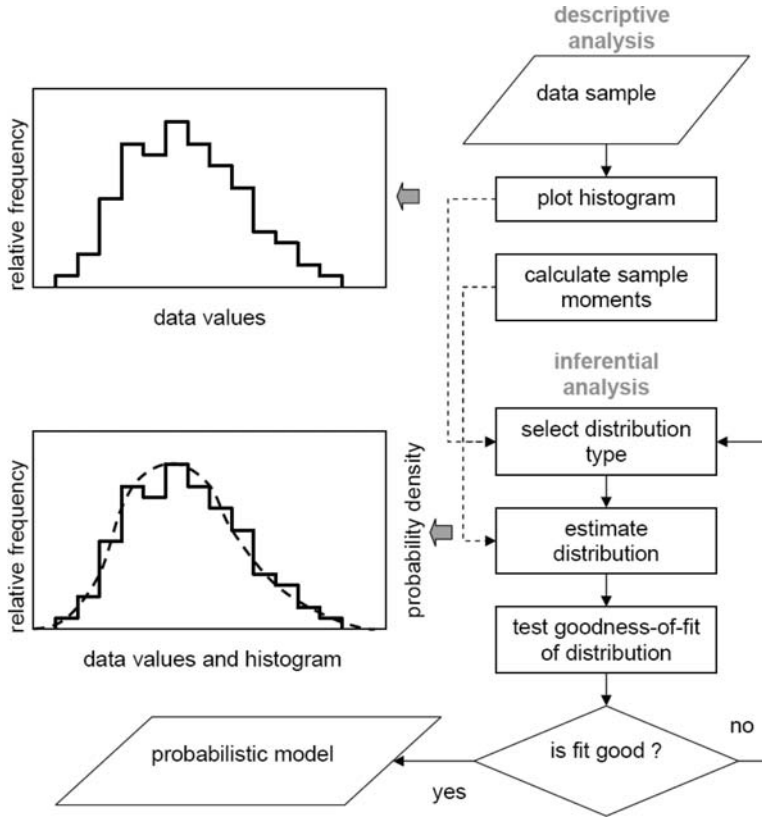


Figure 4. Integrated descriptive and inferential analysis for probabilistic modelling of a random variable.

The *sample variance* of a set of data is the square of the *sample standard deviation* of the set itself. The latter is given by

$$s_{\psi} = \sqrt{\frac{1}{n-1} \sum_{i=1}^n (\psi_i - m_{\psi})^2} \quad (2)$$

The unbiased estimates of the *skewness* and *kurtosis* of a data set are given by, respectively:

$$C_{sk} = \frac{n}{(n-1)(n-2)} \sum_{i=1}^n \left(\frac{\psi_i - m_{\psi}}{s_{\psi}} \right)^3 \quad (3)$$

$$C_{ku} = \left\{ \frac{n(n+1)}{(n-1)(n-2)(n-3)} \sum_{i=1}^n \left(\frac{\psi_i - m_{\psi}}{s_{\psi}} \right)^4 \right\} - \frac{3(n-1)^2}{(n-2)(n-3)} \quad (4)$$

The sample moments as defined above are unbiased estimators of the *distribution moments* of the random variable itself (i.e. the *real* values, calculated on a sample of infinite size).

2.3.1 Statistical estimation error

Any data set is unable to represent a population perfectly because of its size which is always limited in practice. The sample statistics which are calculated from a data set are thus unable to represent the true statistics of the population perfectly, in the sense that: (a) they may be biased; and (b) that there is some

degree of uncertainty in their estimation. Sample statistics, in other words, are affected by *statistical estimation uncertainty*.

If a data set consisting of n elements is assumed to be *statistically independent*, the expected value of the sample mean is equal to the (unknown) population mean; hence, the sample mean is an *unbiased estimator* of the population mean. However, the sample mean has a variance and, consequently, a standard deviation. The latter is given by $s_\psi\sqrt{n}$, in which s_ψ is the sample standard deviation. The coefficient of variation of statistical estimation error is thus defined as

$$\eta_S = \frac{s_\psi}{m_\psi\sqrt{n}} \quad (5)$$

Confidence intervals for the mean can also be calculated (see e.g. Ang & Tang 1975). It is important to note that statistical estimation error decreases with increasing sample size.

2.4 Inferential analysis

The primary goal of inferential analysis is the modelling of a random variable (for which at least one sample is available) by a *probability distribution function*. Such function assigns a level of probability to every interval of the possible values taken by the random variable of interest. Once the type and the parameters of probability distribution have been assigned, the probabilities associated with the interaction of random variables in complex geotechnical systems (e.g. reliability analysis) can be calculated.

A distribution is called *discrete* if the random variable can only attain values from a finite set. A distribution is called *continuous* if the range of the random variable is continuous, i.e. the variable can take any value within a specific interval.

Probability distributions, in principle, may be more or less complex. However, it is advisable, for practical purposes, to refer to probability distributions whose properties are well known. Moreover, experience has shown that a relatively limited set of mathematical functions are able to fit satisfactorily a wide range of observed or assumed distributions (Baecher & Christian 2003).

2.5 Probability distributions

Some among the most commonly used distributions in the geotechnical engineering literature are addressed briefly in the following.

2.5.1 Uniform distribution

The probability density function of the uniform distribution, in which the minimum and maximum bounds $[a]$ and $[b]$ are the parameters, is given by

$$f_\psi(\psi) = \begin{cases} \frac{1}{b-a} & a \leq \psi \leq b \\ 0 & \text{otherwise} \end{cases} \quad (6)$$

Figure 5 shows two uniform distribution functions.

2.5.2 Triangular distribution

The probability density function of a triangular distribution with *location* a , *scale* b and *shape* c is given by

$$f_\psi(\psi) = \begin{cases} \frac{2(\psi-a)}{(b-a)(c-a)} & \text{for } \psi < c \\ \frac{2(b-\psi)}{(b-a)(b-c)} & \text{for } \psi \geq c \end{cases} \quad (7)$$

A triangular distribution is correctly defined only for $\psi > a$ and $c < b$. Figure 6 shows two uniform distribution functions. The mean and standard deviation of a triangular distribution are given by, respectively:

$$\mu = \frac{a+b+c}{3} \quad (8)$$

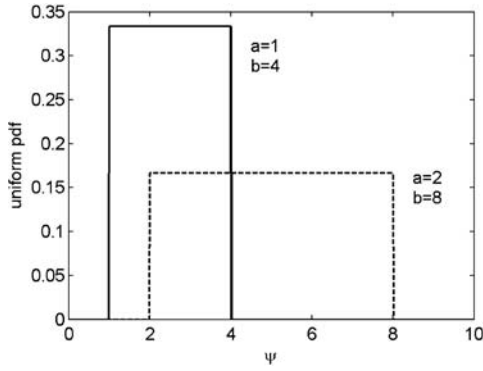


Figure 5. Uniform distribution functions.

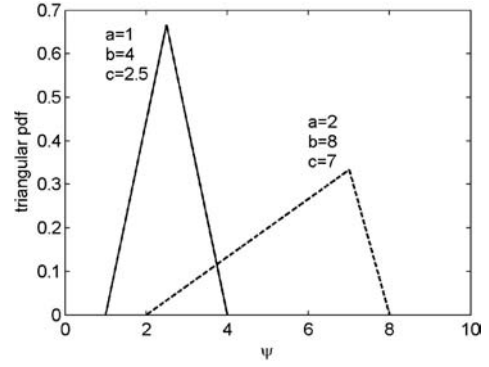


Figure 6. Triangular distribution functions.

$$\sigma = \frac{a^2 + b^2 + c^2 - bc - a(b+c)}{18} \quad (9)$$

Triangular distributions are useful in cases where the upper and lower limits are known, and a most probable value (*mode*) can be identified.

2.5.3 Normal distribution

The normal (or Gaussian) distribution has a probability density function given by

$$f_{\psi}(\psi) = \frac{1}{\sigma\sqrt{2\pi}} \exp\left[-\frac{1}{2}\left(\frac{\psi - \mu}{\sigma}\right)^2\right] \quad (10)$$

in which the parameters of the distribution, μ and σ , are the mean and standard deviation, respectively, of the variable Ψ . The normal distribution is defined in the range $-\infty < \psi < \infty$, and is symmetric around the mean value. While it is one of the most frequently referred to distributions because of its many important properties, it allows for negative values. As most physical properties cannot take negative values, this can provide inconsistencies. Figure 7 shows two normal distribution functions.

2.5.4 Lognormal distribution

A random variable is lognormally distributed if its natural logarithm is normally distributed. The probability density function of a lognormal distribution with parameters $[\lambda]$ and $[\zeta]$ is

$$f_{\psi}(\psi) = \frac{1}{\psi\sqrt{2\pi}\zeta} \exp\left[-\frac{1}{2}\left(\frac{\ln\psi - \lambda}{\zeta}\right)^2\right] \quad (11)$$

and is defined in the range $0 < \psi < \infty$. The distribution parameters, which represent, respectively, the standard deviation and mean of the underlying normal distribution, can be obtained from the mean and standard deviation of the variable Ψ :

$$\zeta = \sqrt{\ln\left(1 + \frac{\sigma^2}{\mu^2}\right)} \quad (12)$$

$$\lambda = \ln\mu - \frac{1}{2}\zeta^2 \quad (13)$$

Figure 8 shows two lognormal distribution functions. The lognormal distribution is employed very frequently because it is consistent with the fact that most physical properties are non-negative.

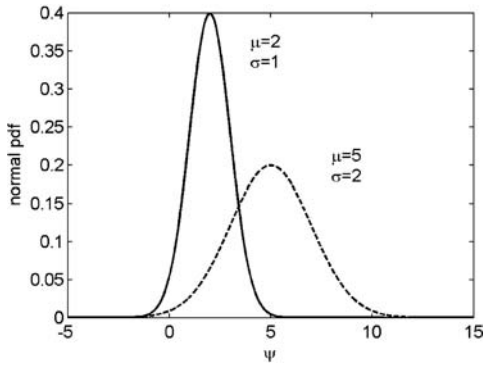


Figure 7. Normal distribution functions.

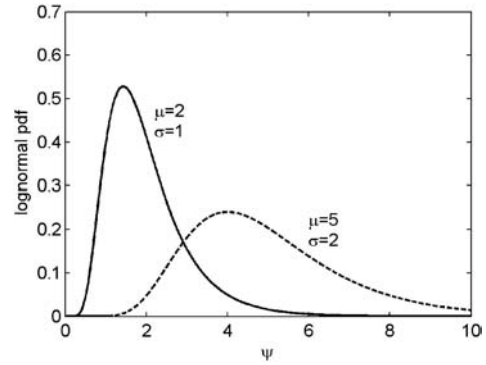


Figure 8. Lognormal distribution functions.

2.5.5 Type-I Pearson beta distribution

A reverse-U type-I Pearson beta distribution is completely identified by four parameters: mean $[\mu]$; standard deviation $[\sigma]$; lower bound $[a]$ and upper bound $[b]$. The probability density function is

$$f_{\psi}(\psi) = C_{\beta} (\psi - a)^{\lambda_1} (b - \psi)^{\lambda_2} \quad (14)$$

in which

$$C_{\beta} = \frac{(\lambda_1 + \lambda_2 + 1)!}{\{\lambda_1! \lambda_2! (b - a)^{\lambda_1 + \lambda_2 + 1}\}} \quad (15)$$

The characteristic parameters of the distribution are given by

$$\lambda_1 = \frac{X_{\beta}^2 (1 - X_{\beta})}{Y_{\beta}^2} - (1 + X_{\beta}) \quad (16)$$

$$\lambda_2 = \frac{\lambda_1 + 1}{X_{\beta}} - (X_{\beta} + 2) \quad (17)$$

in which

$$X_{\beta} = \frac{\mu - a}{b - a} \quad (18)$$

$$Y_{\beta} = \frac{\sigma}{b - a} \quad (19)$$

It must be verified that $\lambda_1 > 0$ and $\lambda_2 > 0$ for a type-I distribution to be defined. Figure 9 shows two type-I beta distribution functions.

The type-I beta distribution is able to represent physical properties because it defines lower and upper bounds and allows for skewness in the probability density function. In principle, such distribution should be used for physical properties as it allows for skewness and upper and lower limits on possible values. However, as shown in § 2.14, normal and lognormal distributions are used more frequently. This is acceptable if the probability of negative values and/or very high values is very small.

2.6 Selection of distribution

The selection of a probability distribution to suitably represent a data set can be made using a variety of approaches and techniques. In the following, the *principle of maximum entropy* and *Pearson's moment-based system* are addressed briefly.

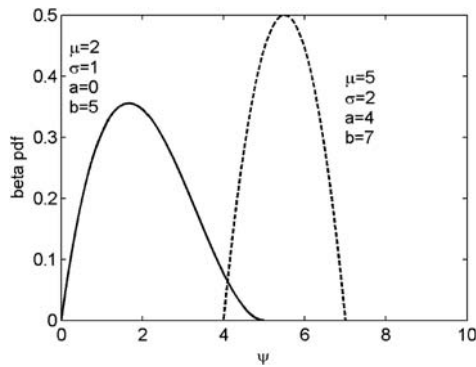


Figure 9. Type-I beta distribution functions.

Table 2. Maximum-entropy criteria and constraints for the selection of a probability distribution.

Mean	Variance	Negative values	Upper and lower bounds	Maximum entropy distribution
Unknown	Unknown	Acceptable	Known	Uniform
Known	Known	Acceptable	Unknown	Normal
Known	Known	Unacceptable	Unknown (upper)	Lognormal
Known	Known	Acceptable	Known	Type-I beta

2.6.1 Principle of maximum entropy

In the context of the selection of a probability distribution, the more general principle of maximum entropy (Jaynes 1978) states that the least biased model that encodes the given information is that which maximises the entropy (i.e. the uncertainty measure) while remaining consistent with this information.

By choosing to use the distribution with the maximum entropy allowed by available information, the most appropriate distribution possible is selected. To choose a distribution with lower entropy would imply unmotivated assumption regarding unavailable information; to choose one with higher entropy would violate the constraints of the available information.

Table 2 provides an example of the various criteria and constraints which may be used in selecting a probability distribution using the maximum entropy principle. For example, according to the principle of maximum entropy, if only the mean and the standard deviation of a random variable are known, and negative values are not acceptable even for small probability levels, a lognormal distribution should be selected. If, for instance, the minimum, maximum, mean value and standard deviation of a random variable are known, the probability distribution to be adopted is a Pearson type-I beta distribution.

2.6.2 Pearson's moment-based system

Pearson developed an efficient system for the identification of suitable probability distributions based on third- and fourth-moment statistics (i.e. skewness and kurtosis, respectively) of a data set. Figure 10 shows Pearson's diagram, whose abscissas are given by, respectively (e.g. Rétháti 1988):

$$\beta_1 = C_{sk}^2 \quad (20)$$

$$\beta_2 = C_{ku} + 3 \quad (21)$$

In principle, it may be seen in Figure 10 that the normal, exponential and uniform distribution are limiting cases of the reverse-U type I beta distribution; these occupy single points in the (β_1, β_2) diagram.

2.7 Distribution fitting

The fitting of a distribution to a data set can rely upon a variety of methods such as *fitting by moments* or *maximum likelihood*. In moment fitting, the first four moments of the data set (mean, variance, skewness and kurtosis) are calculated, and a distribution of the selected type – having the same corresponding moments – is adopted (e.g. Ang & Tang 1975). The reader is referred to Ang & Tang (1975) and Baecher & Christian (2003) for an insight into maximum likelihood approaches.

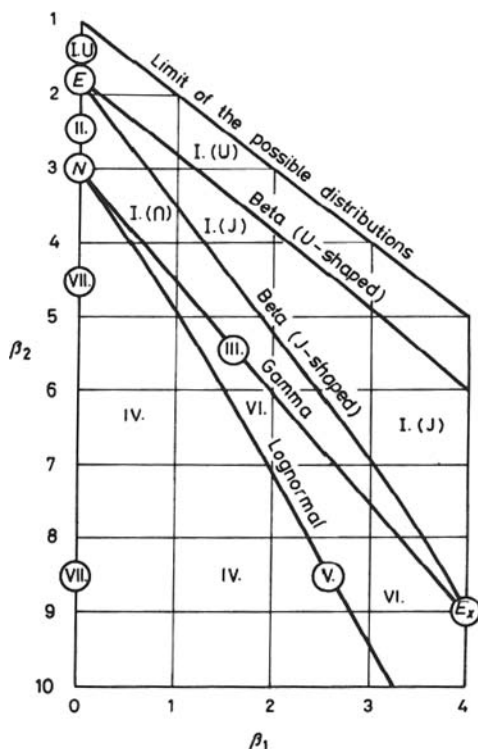


Figure 10. Space of Pearson's probability distributions (from Rétháti 1988).

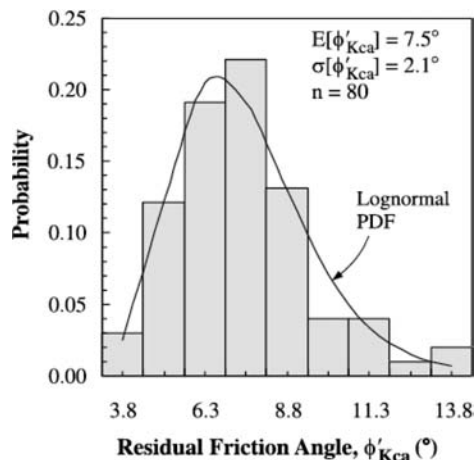


Figure 11. Histogram and fitted log normal distribution for residual friction angle of a Canadian laminated clay shale (El-Ramly et al. 2003).

Figure 11 (El-Ramly et al. 2003) shows an example of a lognormal distribution fitted to the probability histogram of residual friction angle measured from 80 drained direct shear tests for a laminated clay shale at the Syncrude Tailings Dyke in Canada.

2.8 Hypothesis testing

Once a distribution has been fitted, it is advisable to verify its conformity to the data set by testing its *goodness-of-fit* to the available data. A number of approaches to distribution testing are available: these include, for instance, visual inspection of probability plots and hypothesis testing. While the first approach relies on subjective assessment, hypothesis testing is based upon the calculation of appropriate statistics and subsequent comparison with critical values for a reference confidence level.

The normality of residuals, for instance, may be tested statistically using a wide range of normality tests; the reader is referred to the statistical literature. Among the many available methods for testing normality, the Wilk-Shapiro test (Shapiro & Wilk 1965) has been observed to provide good results in comparison with other more widely used tests such as the Kolmogorov-Smirnov test, and is therefore recommended (e.g. Thode 2002). The lognormality of a set of data can be tested for by applying normality tests to the logarithms of the data values.

2.9 Second-moment probabilistic modelling

In *second-moment* approaches, the uncertainty in a random variable can be investigated through its first two moments, i.e. the mean (a *central tendency* parameter) and variance (a *dispersion* parameter). Higher-moment statistics such as skewness and kurtosis are thus not addressed. Second-moment descriptive and inferential modelling of soil parameters are widely used in the geotechnical literature because of their efficiency in transmitting important properties of data sets.

Table 3. Multiplicative coefficient for the estimation of the standard deviation of a normally distributed data set with known range (e.g. Snedecor & Cochran 1989).

n	N_n	n	N_n	n	N_n
2	0.886	11	0.315	30	0.244
3	0.510	12	0.307	50	0.222
4	0.486	13	0.300	75	0.208
5	0.430	14	0.294	100	0.199
6	0.395	15	0.288	150	0.190
7	0.370	16	0.283	200	0.180
8	0.351	17	0.279		
9	0.337	18	0.275		
10	0.325	19	0.271		
		20	0.268		

The *sample coefficient of variation* is obtained by dividing the sample standard deviation by the sample mean. It provides a concise measure of the *relative* dispersion of data around the central tendency estimator:

$$COV_{\psi} = \frac{s_{\psi}}{m_{\psi}} \quad (22)$$

The coefficient of variation is commonly used in geotechnical variability analyses. The advantages are that it is dimensionless and provides a more physically meaningful measure of dispersion relative to the mean. Coefficients of variation of the same physical properties at sites worldwide vary within a relatively narrow range; moreover, they are thought to be independent of the geological age of the soil. This is advantageous as it allows use of literature values with some confidence even at sites for which little or no data may be available (Phoon & Kulhawy 1999a). Harr (1987) provided a “rule of thumb” by which coefficients of variation below 10% are considered to be “low”, between 15% and 30% moderate”, and greater than 30%, “high”.

2.10 Approximate estimation of sample second-moment statistics

It may be necessary, at times, to obtain sample statistics in cases in which other statistics are known but the complete data set is not available. A number of techniques yielding quick, approximate estimates of sample statistics have been proposed.

2.10.1 Approximate estimation of sample standard deviation

For data which can be expected (on the basis of previous knowledge) to be symmetric about its central value, the mean can be estimated as the average of the minimum and maximum values; hence, knowledge of the extreme values would be sufficient. If the range and sample size are known, and if the data can be expected to follow at least approximately a Gaussian (normal) distribution, the standard deviation can be estimated by Eq. (23) (e.g. Snedecor & Cochran 1989), in which the coefficient N_n depends on the size of the sample as shown in Table 3.

$$s_{\psi} \approx N_n (\psi_{\max} - \psi_{\min}) \quad (23)$$

2.10.2 Three-sigma rule

Dai & Wang (1992) stated that a plausible range of possible values of a property whose mean value and standard deviation are known can vary between the mean plus or minus three standard deviations. If it is of interest to assign a value to the standard deviation, the statement can be inverted by asserting that the standard deviation can be taken as one sixth of a plausible range of values. The three-sigma rule does not require hypotheses about the distribution of the property of interest even though its origin relies on the normal distribution.

In case of spatially ordered data, Duncan (2000) proposed the *graphical three-sigma rule method*, by which a spatially variable standard deviation can be estimated. To apply the method, it is sufficient to select (subjectively or on the basis of regression) a best-fit line through the data. Subsequently, the *minimum* and *maximum conceivable bounds lines* should be traced symmetrically to the average line.

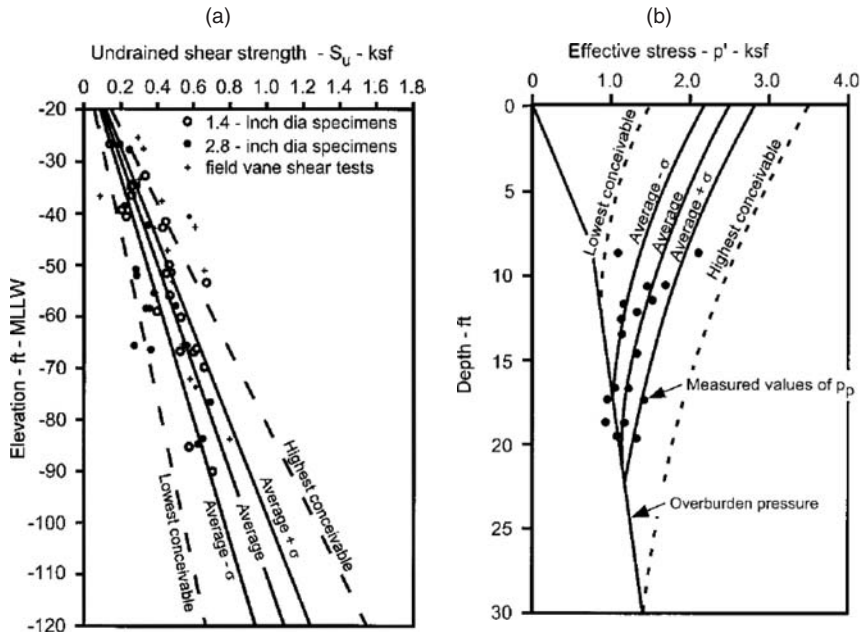


Figure 12. Applications of the graphical three-sigma rule for the estimation of the standard deviation of: (a) undrained shear strength; and (b) preconsolidation stress of San Francisco Bay mud (Duncan 2000).

The *standard deviation lines* can then be identified as those ranging one-third of the distance between the best-fit line and the minimum and maximum lines. Applications of the graphical three-sigma rule (Duncan 2000) are shown in Figure 12.

While the three-sigma rule is simple to implement and allows exclusion of outliers, Baecher & Christian (2003) opined that the method may be significantly unconservative as persons will intuitively assign ranges which are excessively small, thus underestimating the standard deviation. Moreover, for the method to be applied with confidence, the expected distribution of the property should at least be symmetric around the mean, which is not verified in many cases.

2.11 Correlation between random variables

When dealing with more than one random variable, uncertainties in one may be associated with uncertainties in another, i.e. the uncertainties in the two variables may not be independent. Such dependency (which may be very hard to identify and estimate) can be critical to obtaining proper numerical results in engineering applications (e.g. Ang & Tang 1975).

The most common measure of dependence among random variables is the *correlation coefficient*. This measures the degree to which one uncertain quantity varies *linearly* with another.

The correlation coefficient for two uncertain quantities X and Y (represented respectively by two sets of data $x_1 \dots x_n$ and $y_1 \dots y_n$) is defined as the ratio of the covariance of the two variables to the product of the standard deviations of the two sets:

$$\rho(\Psi_1, \Psi_2) = \frac{\sum_{i=1}^n (\psi_{1i} - m_{\psi 1})(\psi_{2i} - m_{\psi 2})}{\sqrt{\sum_{i=1}^n (\psi_{1i} - m_{\psi 1})^2 (\psi_{2i} - m_{\psi 2})^2}} \quad (24)$$

in which $m_{\psi 1}$ and $m_{\psi 2}$ are the sample means of Ψ_1 and Ψ_2 , respectively. The correlation coefficient is non-dimensional, and varies in the range $[-1, +1]$. A higher bound implies a strict linear relation of positive slope (e.g. Figure 13a), while the lower bound attests for a strict linear relation of negative slope. The higher the magnitude, the more closely the data fall on a straight line. The clause of linearity

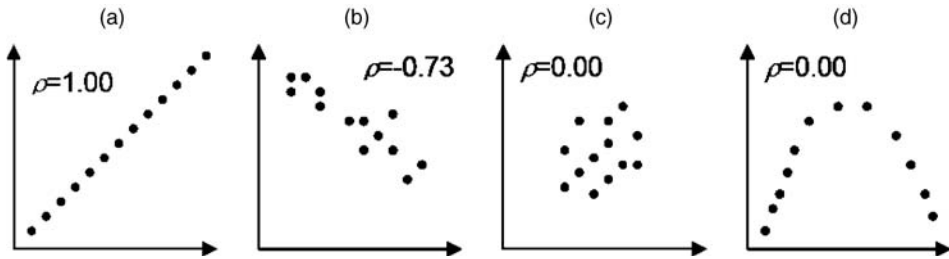


Figure 13. Examples of correlation coefficient between two variables.

should not be overlooked when interpreting the meaning of correlation: two uncertain quantities may be deterministically related to one another, but may have negligible correlation if the relationship is strongly non-linear (e.g. Figure 13d).

2.12 First-order second-moment (FOSM) approximation

In engineering design it is common to use parameters which are functions of measured properties. If design is performed in non-deterministic perspective, it is necessary to estimate the uncertainty in such parameters. This can be achieved by using techniques which process the uncertainty in source (measured) parameters and in the transformation models used to derive the design parameters themselves.

The type and complexity of the techniques to be used in uncertainty-based derivation of design parameters depend mainly upon the level of accuracy required in the design phase and the approach by which uncertainty in input parameters is modelled. It is intuitive that the sophistication of uncertainty characterisation in output variables cannot exceed that of input variables; conversely, it is to be expected that additional uncertainty would be introduced because of uncertainties in the engineering models used to obtain design parameters, as well as by approximations in the uncertainty propagation techniques.

If the uncertainties in measured parameters are modelled probabilistically in the second-moment sense, it is necessary to adopt an uncertainty propagation technique which is compatible with random variables described in such a way. Techniques such as Monte Carlo simulation, for instance, would require assumptions regarding the probability distributions of the measured parameters; this, however, is not compatible with the second-moment approach, in which it is chosen to represent random variables by only the first two statistical moments of the distributions, thereby neglecting all additional information.

First-order second-moment approximation (FOSM) provides an effective means of investigating the propagation of second-moment uncertainties by providing an approximate estimate of the central tendency parameter (e.g. mean) and the dispersion parameter (e.g. standard deviation) of a random variable which is a function of other random variables (e.g. Ang & Tang 1975; Melchers 1999).

By FOSM approximation, errors are introduced because of the fact that higher-order terms in the Taylor series are neglected. In engineering applications making use of second-moment statistics, it should be verified that the magnitude of such errors is small in comparison with the imprecision inherent to second-moment modelling of input parameters and those associated with the transformation model. This is especially important in geotechnical applications, as the coefficients of variation of soil parameters are much larger than those of construction materials routinely used in structural engineering, e.g. concrete and steel.

As an example of FOSM approximation, the calculation of the cone factor Uzielli et al. (2006) based on undrained strength data from triaxial compression tests and on net cone resistance from CPTU tests is shown in Figure 14. The figure shows, at each of the 17 reference measurement depths for which both sets of data were available: (a) mean values and standard deviations for undrained strength; (b) mean values and standard deviations for corrected cone resistance; and (c) mean values and standard deviations for the cone factor. It can be seen that all expected values of the FOSM-estimated cone factor are included within one standard deviation of the mean value in the soil unit.

One very important application of FOSM techniques in the geotechnical literature is the formulation of total uncertainty models, which allow second-moment description of random variables for direct implementation in probabilistic design. It has been explained in §1.4 that the main contributing sources to total uncertainty in a geotechnical design parameter calculated from a set of measured parameters using an empirical, semi-empirical or theoretical transformation model are: (a) the inherent variability of the measured parameters; (b) measurement error in the measured parameters; (c) statistical estimation uncertainty in the measured parameters; and (d) bias and uncertainty in the transformation model.

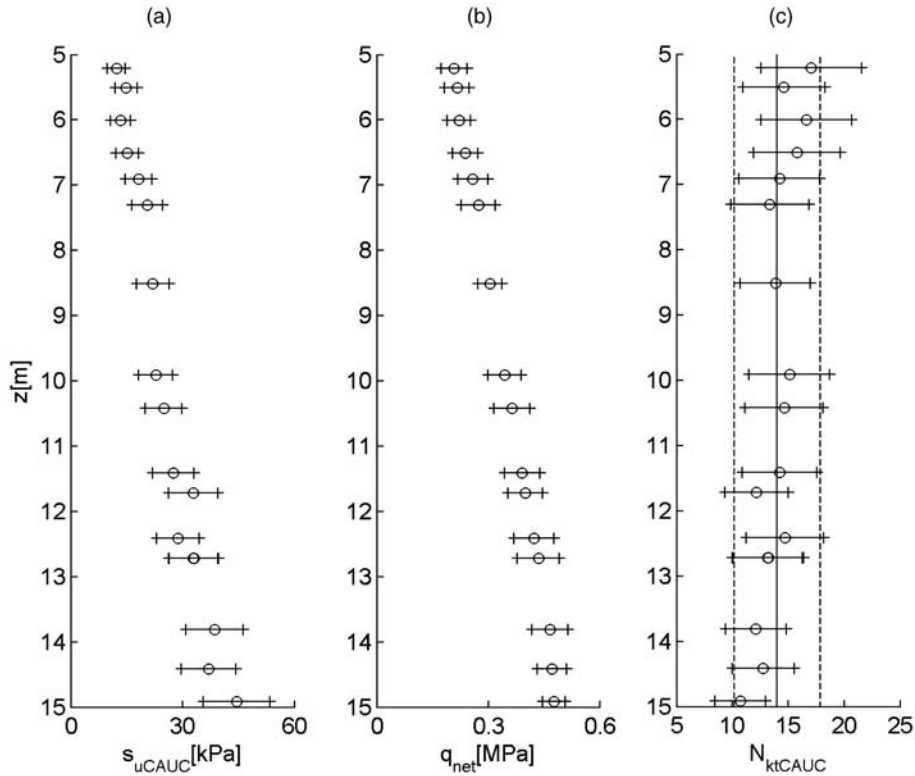


Figure 14. First-order second-moment characterisation of cone factor: (a) triaxial compression (CAUC) – measured undrained shear strength; (b) CPTU net cone resistance; (c) cone factor (Uzielli et al. 2006).

If the measured parameters are modelled in the second-moment sense (i.e. in mean and variance of measurements are calculated), FOSM approximation can be used to transpose such statement to a quantitative framework, i.e. to define a second-moment total uncertainty model. A number of second-moment uncertainty models have been proposed in the geotechnical literature. Phoon & Kulhawy (1999b), for instance, proposed an additive model for the total coefficient of variation of a point design parameter P_D obtained from a single measured property P_M using a transformation model M_t :

$$COV^2(P_D) = COV_{\omega}^2(P_M) + COV_m^2(P_M) + COV_M^2(M_t) \quad (25)$$

in which $COV(P_D)$ is the total coefficient of variation of the design property (neglecting variance reduction effects due to spatial averaging); $COV_{\omega}(P_M)$ is the coefficient of variation of inherent variability of the measured property; $COV_m(P_M)$ is the coefficient of variation of measurement uncertainty of the measured property; and $COV_M(M_t)$ is the coefficient of variation of transformation uncertainty of the transformation model. The interested reader is referred to Ang & Tang (1975) and Phoon & Kulhawy (1999b) for an insight into the application of FOSM techniques for the derivation of total geotechnical uncertainty models.

2.13 Practical difficulties in separating uncertainty components

A vast body of second-moment statistics of geotechnical properties are available in the literature. These data generally pertain to sets of measurements obtained on soil units which were considered to be homogeneous in some way (e.g. “clayey soils”).

It is extremely difficult in practice to evaluate the various sources of uncertainty separately. Consequently, the coefficients of variation of measured parameters usually reported in the geotechnical literature, even though they are termed “coefficient of variation of inherent variability”, refer to *total uncertainty*, i.e. they account for spatial variability, measurement error and statistical estimation error; hence, their values generally overestimate the real magnitude of spatial variability alone. Phoon &

Table 4. Second-moment sample statistics for water content of Edmonton clay (Fredlund & Dahlman 1971).

Mean depth (m)	No. of samples	Mean(%)	Std. dev.(%)	COV (<i>w</i>)
0.6	392	27.4	6.0	0.22
1.2	430	28.9	5.1	0.18
1.8	439	31.2	4.1	0.13
2.4	422	33.0	4.1	0.12
3.0	415	32.8	3.9	0.12
3.6	392	34.7	3.5	0.10
4.2	362	35.0	4.3	0.12
4.8	307	35.5	4.3	0.12
5.4	249	35.5	4.6	0.13
6.0	177	34.8	4.5	0.13

Kulhawy (1999a) defined the coefficient of variation of inherent variability more rigorously as described in §4.8. The calculation of such parameter requires procedures described in later sections of this paper. Very few estimates of the “real” coefficient of variation of inherent variability are available in the literature; these are also reported in §4.8.

If data referring to total uncertainty from a source site are to be used inferentially, it must be verified that it is appropriate to export data to the target site.

The effects of *exogenous factors*, which may influence the magnitudes of a given geotechnical parameter, should be recognised if pertinent. For instance, the undrained strength of a cohesive soil generally varies with depth (even if the soil is compositionally homogeneous) due to increasing overburden stress, overconsolidation effects and other in-situ factors. If the mean and standard deviation of undrained strength are calculated for two soil units which are similar in composition and in stratigraphic location (i.e. their upper interfaces are approximately at the same depth from ground surface), but different in thickness, it may be expected that the mean value of the thicker layer will be greater than that of the other layer.

Such possible bias is related to the existence of spatial trends in data. The issue of spatial trends will be addressed in detail in §3.3. Total variability data from a source site should not be exported to a target site unless the exogenous factors which may affect a soil property of interest are comparable at the two sites. Alternatively, it could be useful and appropriate, in the above example, to address second-moment statistics of stress-normalised strength to eliminate the dependency on the exogenous factor “stress”.

A number of warnings against uncritical exportation of total uncertainty data have been made (e.g. Rétháti 1988; Lacasse & Nadim 1996; Phoon & Kulhawy 1999a). For instance, extreme values and outliers should not be accepted uncritically, as these could strongly bias statistical parameters. Outliers can be identified through statistical procedures such as filtering; however, it is essential that data are also inspected in the light of geotechnical knowledge. The temporal distribution of the collection of data is important. If the data are collected over a time period of 1–2 weeks, the physical characteristics of the soils may be regarded as time-invariant. Over longer periods of time, the scatter in properties of interest may be partly attributed to changes occurring in soils through time. This is particularly true for surficial soil layers, which are most exposed to meteorological variations and to soils in the zone of fluctuation of groundwater level). Also, the standard deviation (and, consequently, the coefficient of variation) generally increases with increasing dimensions of the sampling domain (e.g. areal extension of a site; thickness of a layer). Therefore, it should be verified that the dimensions of the target site are comparable to those of the source site.

2.14 Statistics of soil properties: literature review

The results of second-moment modelling of soil properties are generally provided in tabular form. The compilation of descriptive statistics is not an entirely mechanical procedure. Geotechnical expertise is very important in planning a statistical analysis as to obtain results which may be as significant and useful as possible for characterisation and design. Given the above considerations, it is deemed useful to report two examples of best-practice statistical processing of data from the geotechnical literature. Subsequently, the results of a literature review of second-moment statistics are provided.

2.14.1 Second-moment statistics: best-practice examples

Fredlund & Dahlman (1971) provided tabulated values of second-moment sample statistics for Edmonton clay. In their results for water content (Table 4), they subdivided the soil unit into sub-units of thickness

Table 5. Number of laboratory tests of soils at Szeged (adapted from Rétháti 1988).

Soil type	Sample type	w	I_P, I_C	e, S_r, γ	q_u
S1	T1	78	64	—	—
	T2	56	37	54	19
	T3	24	8	25	21
S2	T1	50	47	—	—
	T2	85	44	85	29
	T3	9	7	9	6
S3	T1	371	340	—	—
	T2	531	281	538	227
	T3	46	27	46	35
S4	T1	375	356	—	—
	T2	512	197	513	299
	T3	129	70	129	108
S5	T1	189	186	—	—
	T2	52	9	52	32
	T3	108	57	108	85

Table 6. Second-moment sample statistics of Szeged soils (adapted from Rétháti 1988).

	Soil type	w (%)	w_L (%)	w_P (%)	I_P (%)	I_C	e	S_r	γ (kN/m ³)	q_u (kPa)
Mean	S1	31.1	44.5	24.2	20.6	0.62	0.866	0.917	18.8	144
	S2	22.3	32.3	19.4	13.0	0.82	0.674	0.871	19.7	125
	S3	24.5	32.2	20.7	11.5	0.63	0.697	0.902	19.7	173
	S4	28.3	54.0	24.2	29.9	0.86	0.821	0.901	19.3	219
	S5	28.6	52.8	25.4	27.4	0.82	0.823	0.905	19.2	184
COV	S1	0.30	0.39	0.34	0.57	0.49	0.26	0.10	0.07	0.45
	S2	0.18	0.13	0.12	0.37	0.37	0.13	0.12	0.04	0.38
	S3	0.15	0.13	0.11	0.36	0.47	0.13	0.11	0.04	0.59
	S4	0.17	0.21	0.14	0.34	0.20	0.11	0.10	0.03	0.53
	S5	0.17	0.27	0.15	0.44	0.32	0.12	0.10	0.03	0.54

0.6 m. It is possible to appreciate the differences in mean values and coefficients of variation among the sub-units. If the 6-metre layer had been treated as a whole, some variability information would have been lost. The high numerosity of the samples, which reduces the statistical estimation error, should also be noted.

In a comprehensive statistical study of the subsoil of the town of Szeged in Hungary performed in 1978, Rétháti and Ungár (see Rétháti 1988) obtained second-moment statistics for soil physical characteristics for 5 soil types (S1: humous-organic silty clay; S2: infusion loess, above groundwater level; S3: infusion loess, below groundwater level; S4: clay; S5: clay-silt) using the results of approximately 11000 tests from 2600 samples collected over 25 years.

The number of laboratory tests was reported (Table 5) by sample types (T1: partly disturbed samples placed in metal boxes; T2: core samples taken from small diameter boreholes; T3: core samples taken from large diameter boreholes) and the physical characteristics tested (water content w , plasticity index I_P , consistency index I_C , void ratio e , degree of saturation S_r , unit weight γ , unconfined compression strength q_u). Second-moment statistics are shown in Table 6. Recent compilations of soil statistics include Sillers & Fredlund (2001) and Chin et al. (2006).

2.14.2 Second-moment statistics: literature review

The results of a literature review of second-moment statistics in the form of *mean value*, *coefficient of variation* and *coefficients of correlation* between properties are reported hereinafter, as well as discussions on suitable *probability distributions* for a number of soil properties.

The data presented hereinafter are useful for reference purposes; they should not be used uncritically in design because statistics of those geotechnical parameters which are related to in-situ state are significantly dependent upon the in-situ state (or simulation thereof in the laboratory). For these parameters, it is difficult to identify *typical* values. Also, in geotechnical engineering it is often possible to measure the same parameter using two or more testing methods and/or procedures. Different testing procedures

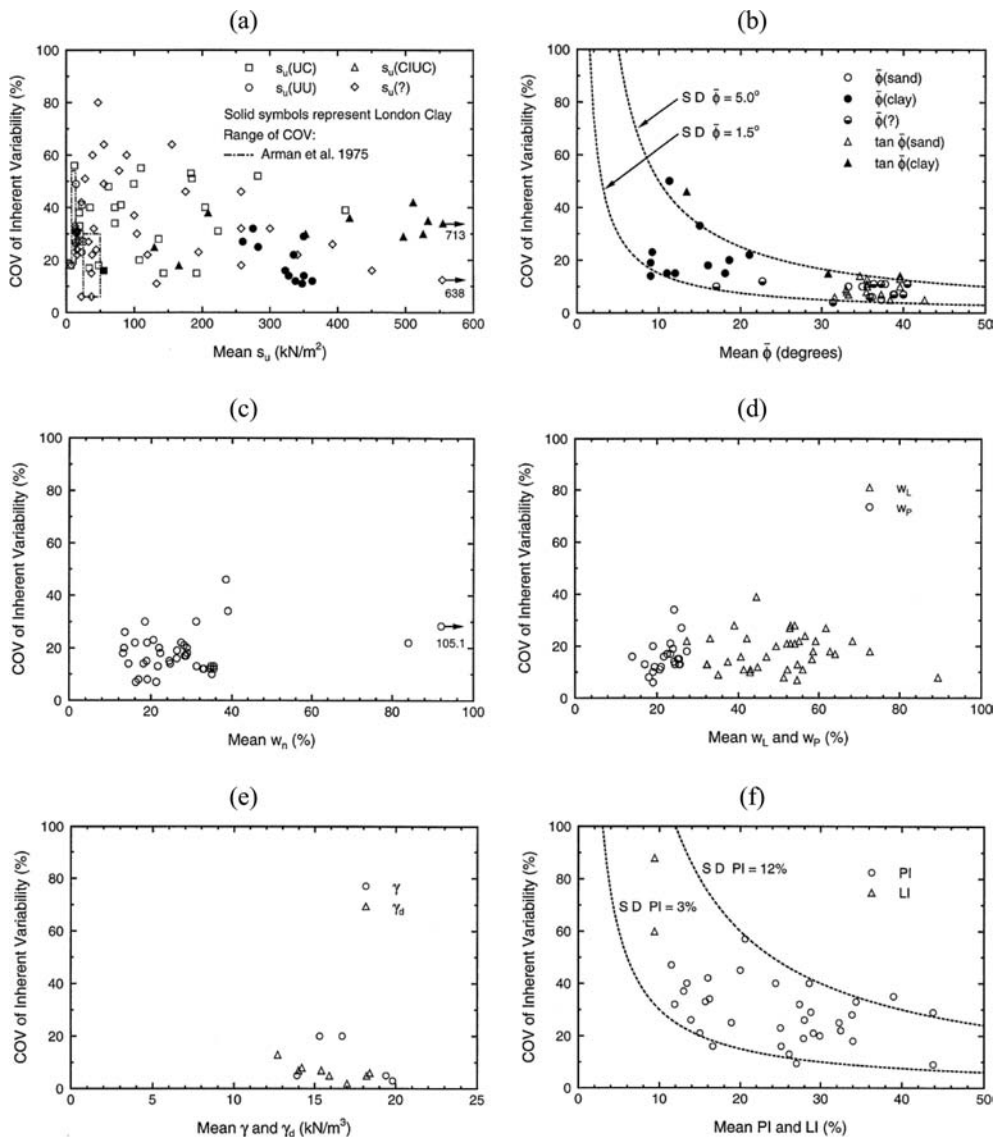


Figure 15. Coefficient of variation vs. mean values for laboratory measurements: (a) undrained shear strength; (b) friction angle; (c) natural water content; (d) liquid and plastic limits; (e) unit weight; and (f) liquidity and plasticity indices (Phoon & Kulhawy 1999a).

are generally characterised by different testing uncertainty. Moreover, using more than one testing procedure will result in different measured values because the measurement occurs in a different way, e.g. measurement of undrained strength by triaxial compression testing involves different deformations in soils than, for instance, direct shear testing, triaxial extension or vane testing. Hence, the testing method should be specified when reporting statistics from a source site. Lastly, it is generally not possible to evaluate the degree of homogeneity in the soil units from which the statistics are calculated. If such information is not provided, descriptive and inferential statistics will be misleading.

Figure 15 (Phoon & Kulhawy 1999a) reports the results of an extensive literature review of coefficients of variations of inherent variability for some laboratory-measured geotechnical properties, namely: (a) undrained shear strength; (b) friction angle; (c) natural water content; (d) liquid and plastic limits; (e) unit weight; and (f) liquidity and plasticity indices. In the diagrams, coefficients of variation of inherent variability are plotted against mean value. Each point represents a data set.

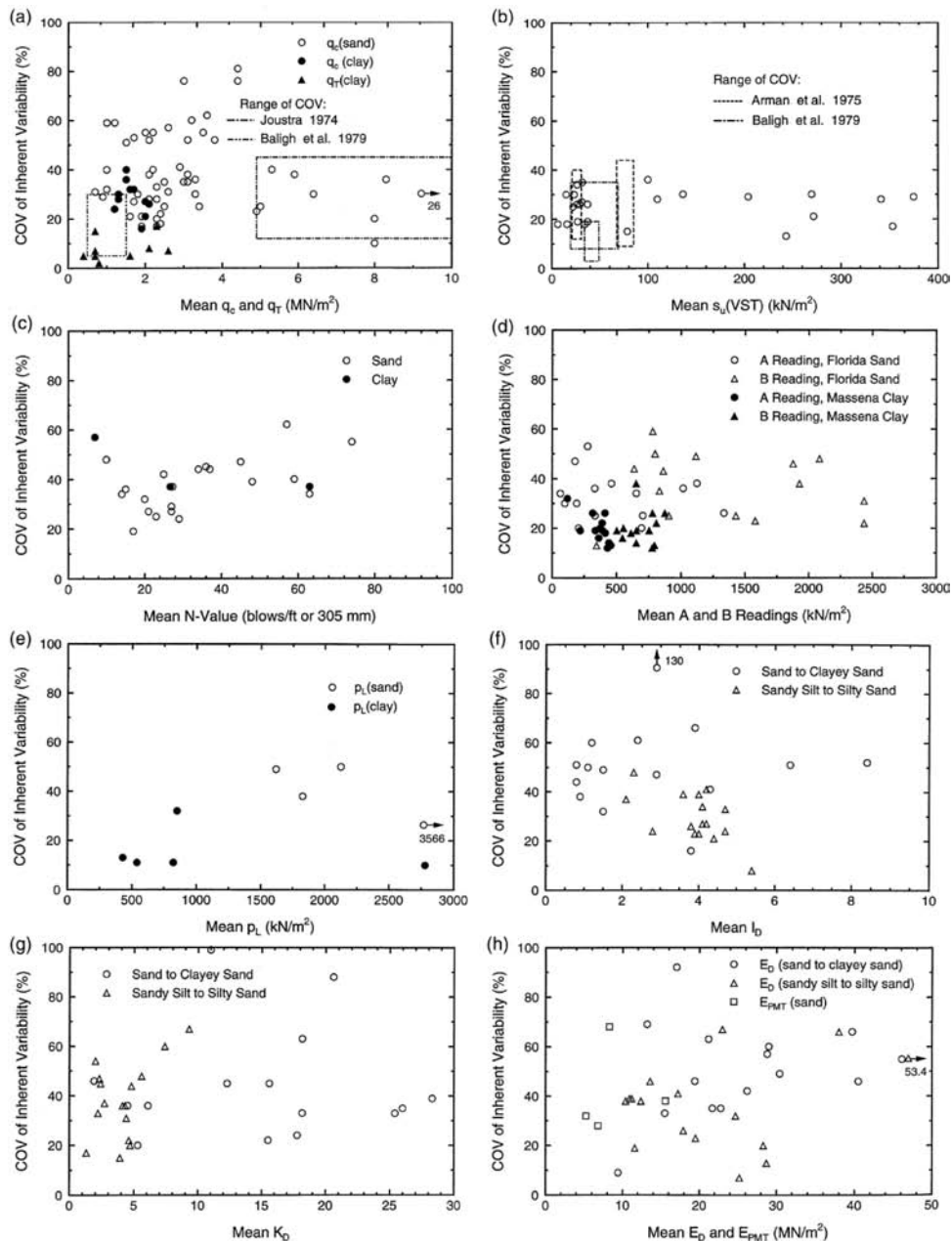


Figure 16. Coefficient of variation vs. mean values for in-situ measurements: (a) cone tip resistance from CPT; (b) undrained shear strength from field vane testing; (c) SPT blow counts; (d) A and B readings from DMT testing; (e) pressuremeter limit pressure; (f) dilatometer material index; (g) dilatometer horizontal stress index; (h) dilatometer modulus and pressuremeter modulus (Phoon & Kulhawy 1999a).

Figure 16 reports the coefficients of variations of inherent variability for some in-situ testing parameters (Phoon & Kulhawy 1999a): (a) cone tip resistance from CPT; (b) undrained shear strength from field vane testing; (c) SPT blow counts; (d) A and B readings from DMT testing; (e) pressuremeter limit stress; (f) dilatometer material index; (g) dilatometer horizontal stress index; (h) dilatometer modulus and pressuremeter modulus.

Table 7. Approximate guidelines for second-moment statistics of soil parameters.

Test type	Property*	Soil type	Mean	COV (%)
Lab strength	s_u (UC)	Clay	10–400 kN/m ²	20–55
	s_u (UU)	Clay	10–350 kN/m ²	10–30
	s_u (CIUC)	Clay	150–700 kN/m ²	20–40
	ϕ'	Clay & sand	20–40°	5–15
CPT	q_T	Clay	0.5–2.5 MN/m ²	<20
	q_c	Clay	0.5–2.0 MN/m ²	20–40
		Sand	0.5–30.0 MN/m ²	20–60
VST	s_u (VST)	Clay	5–400 kN/m ²	10–40
SPT	N	Clay & sand	10–70 blows/ft	25–50
DMT	A reading	Clay	100–450 kN/m ²	10–35
		Sand	60–1300 kN/m ²	20–50
	B reading	Clay	500–880 kN/m ²	10–35
		Sand	350–2400 kN/m ²	20–50
	I_D	Sand	1–8	20–60
	K_D	Sand	2–30	20–60
	E_D	Sand	10–50 MN/m ²	15–65
PMT	p_L	Clay	400–2800 kN/m ²	10–35
		Sand	1600–3500 kN/m ²	20–50
	E_{PMT}	Sand	5–15 MN/m ²	15–65
Lab index	w_n	Clay & silt	13–100%	8–30
	w_L	Clay & silt	30–90%	6–30
	w_P	Clay & silt	15–25%	6–30
	I_P	Clay & silt	10–40%	**
	I_L	Clay & silt	10%	**
	γ, γ_d	Clay & silt	13–20 kN/m ³	<10
	D_R	Sand	30–70%	10–40*** 50–70****
Lab. consolidation	C_c	Not reported	–	10–37
	p'_c	Not reported	–	10–35
	OCR	Not reported	–	10–35
Not reported	k	Saturated clay	–	68–90
		Partly saturated clay	–	130–240
Not reported	c_v	Not reported	–	33–68
Not reported	n	All soil types	–	7–30
	e	All soil types	–	7–30
	e_0	All soil types	–	7–30

* s_u = undrained shear strength; UC = unconfined compression test; UU = unconsolidated-undrained triaxial compression test; CIUC = consolidated isotropic undrained triaxial compression test; ϕ' = effective stress friction angle; q_T = corrected cone tip resistance; q_c = cone tip resistance; VST = vane shear test; N = standard penetration test blow count; A and B readings, I_D , K_D and E_D = dilatometer A and B readings, material index, horizontal stress index and modulus; p_L and E_{PMT} = pressuremeter limit stress and modulus; w_n = natural water content; w_L = liquid limit; I_P = plasticity index; I_L = liquidity index; γ and γ_d = total and dry unit weights; D_R = relative density; C_c = compression index; p'_c = preconsolidation pressure; OCR = overconsolidation ratio; k = permeability coefficient (direction not specified); c_v = coefficient of vertical consolidation; n = porosity; e = void ratio; e_0 = initial void ratio.

** COV = (3–12%)/mean.

*** Total variability for direct method of determination.

**** Total variability for indirect determination using SPT values.

Table 7 reports typical ranges of mean values and coefficients of variation (accounting for inherent variability, measurement error and estimation error) of laboratory and in-situ testing parameters. Especially those results for which testing method and soil type are not reported should be viewed with extreme caution. Details and references to original sources can be found in Phoon et al. (1995),

Table 8. Correlation matrix for Oroville dam data on impervious borrow material (Holtz & Krizek 1971).

Parameter*	% gr.	% sd.	w_L	w_P	I_P	G_S	γ_d	w	S_r	c'	ϕ'
% gr.	1.00										
% sd.	-0.84	1.00									
w_L	0.13	-0.22	1.00								
w_P	-0.61	0.22	0.57	1.00							
I_P	0.71	-0.47	0.68	-0.19	1.00						
G_S	0.91	-0.62	-0.15	-0.85	0.56	1.00					
γ_d	0.61	-0.34	-0.61	-0.93	0.08	0.82	1.00				
w	-0.68	0.37	0.40	0.88	-0.29	-0.86	-0.91	1.00			
S_r	-0.19	0.35	-0.66	-0.38	-0.46	0.02	0.45	-0.11	1.00		
c'	-0.13	0.40	-0.13	-0.22	0.04	0.07	0.14	0.00	0.67	1.00	
ϕ'	0.44	-0.55	-0.47	-0.41	-0.19	0.39	0.63	-0.52	0.13	-0.49	1.00

Table 9. Correlation matrix for Oroville dam combined data on impervious and pervious borrow material (Holtz & Krizek 1971).

Parameter*	% gr.	% sd.	w_L	w_P	I_P	G_S	γ_d	w	S_r	c'	ϕ'
% gr.	1.00										
% sd.	-0.95	1.00									
w_L	-0.52	0.41	1.00								
w_P	-0.36	0.23	0.91	1.00							
I_P	-0.59	0.53	0.85	0.56	1.00						
G_S	0.32	-0.17	0.06	0.09	0.24	1.00					
γ_d	0.75	-0.59	-0.61	-0.50	-0.58	0.51	1.00				
w	-0.85	0.70	0.64	0.48	0.66	-0.38	-0.93	1.00			
S_r	-0.77	0.69	0.61	0.39	0.72	0.08	-0.51	0.75	1.00		
c'	0.37	-0.38	-0.10	0.00	-0.21	-0.06	0.26	-0.34	-0.28	1.00	
ϕ'	0.72	-0.63	-0.68	-0.49	-0.75	0.18	0.82	-0.81	-0.65	0.10	1.00

* % gr. = percent gravel; % sd. = percent sand; w_L = liquid limit; w_P = plastic limit; I_P = plasticity index; G_S = specific gravity; γ_d = dry unit weight; w = water content; S_r = degree of saturation; c' = effective cohesion; ϕ' = effective friction angle.

Kulhawy & Trautmann (1996), Lacasse & Nadim (1996), Phoon & Kulhawy (1999a) and Jones et al. (2002).

2.15 Correlation in soil properties: literature review

The quantification of correlation between two or more soil properties is important in the context of probabilistic geotechnical approaches, as it provides a more realistic assessment of uncertainty in design parameters which are functions of the properties themselves. A full literature review is hardly feasible. Literature values for the linear correlation coefficient are site-specific and should not be exported uncritically. Selected results are shown hereinafter primarily to highlight the dependence of correlation values from a number of factors. For instance, dependence from soil type is well established. The testing method used to obtain a given parameter generally affects the numerical value of the parameter itself (e.g. undrained strengths measured on the same soils by triaxial compression, direct shear or triaxial extension are different because of the different failure mechanisms involved). Hence, it is to be expected that, as in the case of sample statistics, correlation will also depend from the testing method to some extent. It is also important that soil units from which statistics are to be calculated are homogeneous. Magnan & Bagheri (1982) provided an exhaustive collection of correlation values.

Holtz & Krizek (1971) compiled correlation matrices for a number of soil properties of pervious materials (primarily clean sands, gravels and boulders) and impervious materials (clayey gravels) from the Oroville Dam area in California, measured by laboratory testing. Table 8 illustrates the correlation matrix for impervious materials; Table 9 illustrates the correlation matrix for the combined data (pervious and impervious materials). Comparing Tables 8 and 9, it is seen that in many cases the magnitude of correlation varies significantly (in some cases the sign is even inverted): for instance, the correlation

Table 10. Correlation matrix for Szeged soil S1: humorus-organic silty clay (adapted from Rétháti 1988).

Parameter*	w	w_L	w_P	I_P	I_C	e	S_r	γ
w	1.00							
w_L	0.61	1.00						
w_P	0.76	0.77	1.00					
I_P	0.33	0.89	0.39	1.00				
I_C	-0.23	0.45	0.36	0.39	1.00			
e	0.85	0.39	0.40	0.29	-0.36	1.00		
S_r	-0.04	0.04	0.14	-0.02	0.05	-0.29	1.00	
γ	-0.44	-0.05	-0.09	-0.02	0.30	-0.58	0.34	1.00

Table 11. Correlation matrix for Szeged soil S2: infusion loess, above groundwater level (adapted from Rétháti 1988).

Parameter*	w	w_L	w_P	I_P	I_C	e	S_r	γ
w	1.00							
w_L	0.06	1.00						
w_P	0.36	0.08	1.00					
I_P	-0.13	0.86	-0.44	1.00				
I_C	-0.59	0.20	0.33	0.02	1.00			
e	0.72	0.01	0.00	0.01	-0.55	1.00		
S_r	0.62	0.00	0.22	-0.12	0.38	-0.04	1.00	
γ	-0.33	0.14	0.03	0.11	0.26	-0.85	0.46	1.00

Table 12. Correlation matrix for Szeged soil S3: infusion loess, below groundwater level (adapted from Rétháti 1988).

Parameter*	w	w_L	w_P	I_P	I_C	e	S_r	γ
w	1.00							
w_L	0.26	1.00						
w_P	0.35	0.33	1.00					
I_P	0.07	0.87	-0.12	1.00				
I_C	-0.69	0.29	0.11	0.24	1.00			
e	0.65	0.05	0.00	0.05	-0.52	1.00		
S_r	0.46	0.17	0.18	0.09	-0.30	-0.26	1.00	
γ	-0.11	0.08	-0.03	0.10	0.10	-0.44	0.33	1.00

* w = water content; w_L = liquid limit; w_P = plastic limit; I_P = plasticity index; I_C = consistency index; e = void ratio; S_r = degree of saturation; γ_d = dry unit weight.

between liquid limit and gravel fraction is 0.13 for the impervious materials, but becomes -0.52 if the combined data are considered.

Rétháti (1988) reported matrices of correlation coefficients for Szeged (see also §2.14.1) soil types S1 (Table 10), S2 (Table 11) and S3 (Table 12). If Tables 10, 11 and 12 are compared, it is seen that some of the correlation values vary little between soil type (e.g. liquid limit vs. plasticity index). On the contrary, a number of values are very different. Such is the case, for instance, for water content vs. liquid limit, degree of saturation vs. consistency index and unit weight vs. void ratio. Even more significant is the observation that there are substantial variations in correlation coefficients between S2 and S3, as these soil units refer to the same soil type (infusion loess), and the distinction is only between the stratigraphic depth with respect to groundwater level (S2 are above; S3 are below). While these variations can be explained based on geotechnical considerations (i.e. the expectable difference in the degree of saturation and void ratio between S2 and S3), it is important to remark that they are appreciable as a consequence of a correct choice in the definition of soil units: if S2 and S3 had been classified solely on the basis of soil type, only “lumped”, less informative and possibly unconservative correlation values for the infusion loess could have resulted.

For best-practice, if it is not possible to collect data and calculate the correlation coefficient using Eq. (24), maximum care should be taken as to select literature values which: (a) pertain to soil units which

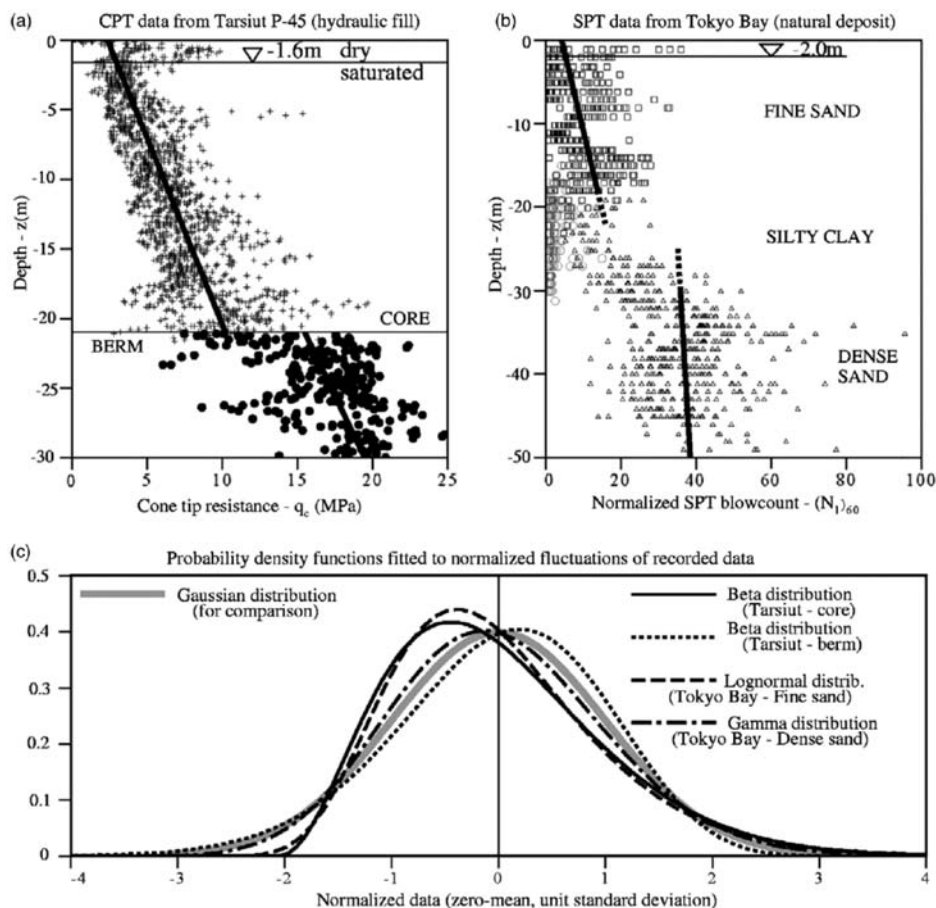


Figure 17. Dependence of probability distributions from in-situ state and soil type (Popescu et al. 1998).

are sufficiently homogeneous in terms of composition, in-situ state and mechanical behaviour; (b) refer to soil types which are affine to the ones under investigation; and (c) are obtained using the same testing methods.

The correlation between effective cohesion and effective friction angle is perhaps the most widely referred to in literature applications, as it enters important analyses such as foundation bearing capacity and slope stability. A comparative literature review yields the following ranges of values for such correlation coefficient: $\rho = -0.47$ (Wolff 1985); $-0.49 \leq \rho \leq -0.24$ (Yucemen et al. 1973); $-0.70 \leq \rho \leq -0.37$ (Lumb 1970); $\rho = -0.61$ (Cherubini 1997). In absence of specifically calculated data, a parametric approach using $-0.75 \leq \rho \leq -0.25$ can be used for practical applications.

2.16 Probability distributions for soil properties: literature review

Different probability distribution models have been selected, even for the same soil property, by different authors. This suggests that distributions are site- and parameter-specific, and that there is no universally "best" distribution for soil properties. In-situ effects, which may result in a spatial trend, may also be relevant. For instance, based on cone penetration data from artificial and natural deposits, Popescu et al. (1998) observed that the distribution of soil strength in shallow layers were prevalently positively skewed, while for deeper soils the corresponding distributions tended to follow more symmetric distributions. This is shown in Figure 17, which also reports the different distributions which were selected for the various soil units.

Despite the substantial data set-dependence of best-fit probability distributions, a number of literature results are reported in the following.

Table 13. Results of Corotis et al. (1975) investigation on probability distributions (in %).

	N	LN	Other
w	67	33	—
w_L	67	33	—
w_p	—	100	—
e	67	—	33
c_v	67	33	—

Table 14. Probability distributions for different soil properties (adapted from Lacasse & Nadim 1996).

Soil property	Soil type	Distribution
Cone resistance	Sand	LN
	Clay	N/LN
Undrained shear strength	Clay (triaxial tests)	LN
	Clay (index tests)	LN
	Clayey silt	N
Stress-normalised undrained shear strength	Clay	N/LN
Plastic limit	Clay	N
Submerged unit weight	All soils	N
Friction angle	Sand	N
Void ratio, porosity	All soils	N
Overconsolidation ratio	Clay	N/LN

Corotis et al. (1975) investigated whether a number of properties of three groups of soils could be described by the normal or lognormal distribution. The Kolmogorov-Smirnov goodness-of-fit test was applied to data sets from the three soil types. The percentage of data sets to which the normal (N) or lognormal (LN) distributions could be fit with sufficient confidence is shown in Table 13.

Lacasse & Nadim (1996) reported the results of a review of probability distribution selection for some soil properties; these are shown in Table 14. It should be noted that best-fit probability distributions may also depend on soil type.

As an example of probability distribution selection using Pearson's moment-based system, the results reported in Rétháti (1988) for Szeged soils are shown in Table 15. The parameters β_1 and β_2 are calculated using Eq. (20) and Eq. (21), respectively. The points are plotted on Pearson's chart (also shown in Figure 10) as illustrated in Figure 18.

With reference to Figure 10 and Figure 18, a number of observations can be made: (a) the consistency index is highly symmetric (i.e. has a small skewness), and can therefore be approximated for 3 of 5 soil units using a normal distribution; (b) a distribution type of general validity cannot be adopted for void ratio; (c) the degree of saturation is highly non-symmetric and can be approximated using a J-shaped and a reversed U-shaped beta distribution.

As a general observation, points corresponding to the same property for different soil units generally plot in different areas of the chart; this reflects the influence of soil type and in-situ state on data distribution. Hence, it is difficult to associate a specific probability distribution to a soil property *a priori*.

2.17 Probabilistic slope stability: literature examples

Second-moment probabilistic slope stability analysis allows assessment of the probability of a pre-defined type of failure. The reliability index can be calculated using a variety of approaches, such as Monte Carlo simulation or First-Order Reliability Method (FORM). Overviews of second-moment probabilistic slope stability investigations are provided in El-Ramly et al. (2002) and Nadim et al. (2005).

Griffiths & Fenton (2004) conducted a probabilistic investigation on the simplified cohesive slope model shown in Figure 19. Initially, they obtained the deterministic factor of safety from limit-equilibrium methods as a function of the normalised, dimensionless undrained shear strength (Figure 20).

The probabilistic approach made use of Monte Carlo simulation (essentially repeated realisations of deterministic limit-equilibrium analyses using strength values sampled from an assumed distribution, in this case lognormal), and allowed to establish the relationship between deterministic factor of safety and

Table 15. β_1 and β_2 values for selecting probability distribution using Pearson's chart for the physical characteristics of Szeged soils (adapted from Rétháti 1988).

Soil		w	w_L	w_P	I_P	I_C	e	S_r	γ	q_u
S1	β_1	2.76	4.12	6.81	1.93	0.13	6.50	7.90	1.28	0.02
	β_2	7.39	8.10	12.30	4.92	3.34	11.19	14.14	3.98	1.86
S2	β_1	0.01	0.74	0.34	0.49	0.02	0.01	0.86	0.09	0.94
	β_2	3.45	3.43	3.27	2.69	3.17	2.67	3.39	3.87	3.93
S3	β_1	0.03	0.96	0.14	0.85	0.00	1.30	3.17	2.89	5.06
	β_2	7.62	5.13	4.32	4.46	3.31	5.52	7.38	11.59	10.72
S4	β_1	0.05	0.34	0.13	0.64	0.03	0.13	3.20	1.06	4.80
	β_2	7.17	3.19	3.47	3.57	4.61	4.03	7.35	8.14	10.95
S5	β_1	1.10	0.02	2.92	0.00	0.98	0.36	2.89	0.10	2.72
	β_2	6.70	2.31	9.15	2.17	5.13	4.14	6.50	4.94	6.74

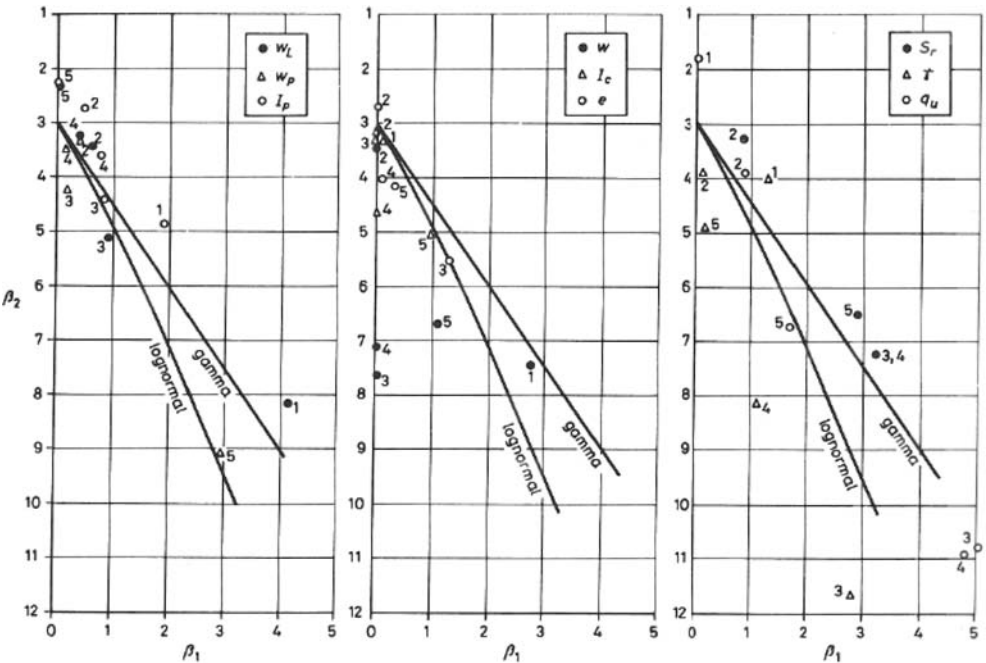


Figure 18. β_1 , β_2 value pairs for physical characteristics of Szeged soils plotted by soil unit (1–5) in Pearson's chart (Rétháti 1988).

probability of failure (shown in Figure 21 for different values of the coefficient of variation of undrained shear strength). A key to understanding the results lay in the observation that if the *median value* of the lognormal distribution of normalised strength was smaller than 0.17, increasing the coefficient of variation of the distribution (i.e. increasing standard deviation or decreasing mean) resulted in decreasing probability of failure; on the contrary, for median values exceeding 0.17, probability of failure increased with increasing coefficient of variation of the distribution (Figure 22).

Nadim & Lacasse (1999) demonstrated the application of FORM to undrained slope stability analysis. They evaluated the factor of safety and probability of failure of a slope (shown in Figure 23) consisting of 2 clay layers with assessed large-scale average property distributions under static and seismic loading (Table 16). Stability analysis relied on Bishop's method; hence, circular slip surfaces were identified. Model uncertainty (i.e. bias and dispersion) was also modelled with second-moment parameters.

Application of FORM showed that the critical surface with the lowest deterministic safety factor is not the critical surface that has the highest probability of failure. Slip surface 1, which cuts through both soil layers, has the lowest deterministic safety factor. However, due to spatial averaging effects,

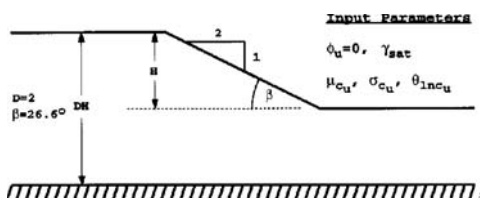


Figure 19. Cohesive slope test problem (Griffiths & Fenton 2004).

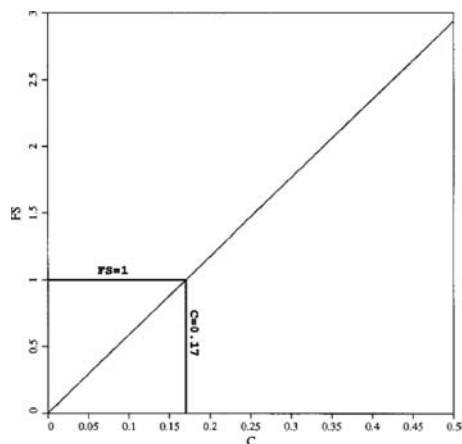


Figure 20. Relationship between factor of safety and normalised strength for the cohesive model slope using limit-equilibrium methods (Griffiths & Fenton 2004).

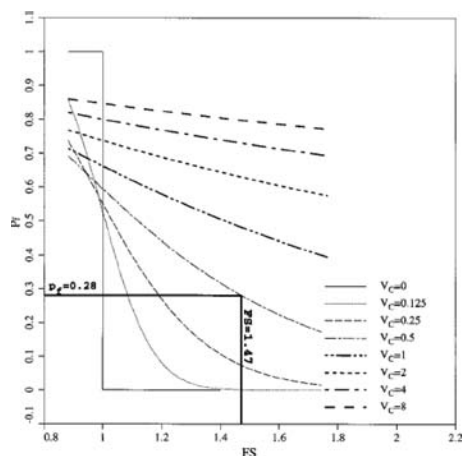


Figure 21. Probability of failure versus factor of safety for the cohesive model slope (Griffiths & Fenton 2004).

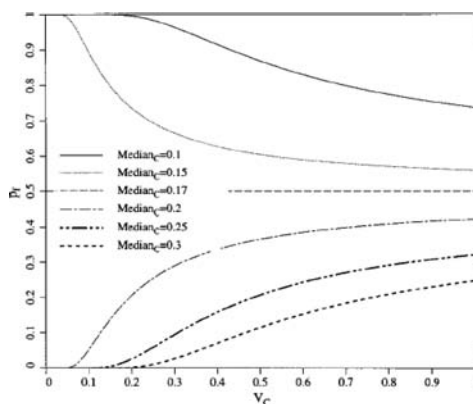


Figure 22. Probability of failure versus coefficient of variation for different values of the median of the log-normal distribution of normalised strength (Griffiths & Fenton 2004).

the uncertainty in the computed safety factor for slip surface 1 is significantly less than that for slip surface 2, which only cuts through the top layer (Figure 24).

Low (2003) proposed a spreadsheet-based methodology for first-order reliability method reliability-based slope stability evaluation using the generalised method of slices. The methodology allows second-moment formulation of soil properties (cohesion, friction angle and unit weight) and performs an iterative numerical search for the minimum-reliability slip surface, which is generally different (as shown in the previous example) from the minimum factor-of-safety surface. Figure 25 shows an example application of the method.

3 SPATIAL CORRELATION ANALYSIS

Second-moment statistics alone are unable to describe the spatial variation of soil properties, whether measured in the laboratory or in-situ. Two sets of measurements may have similar second-moment

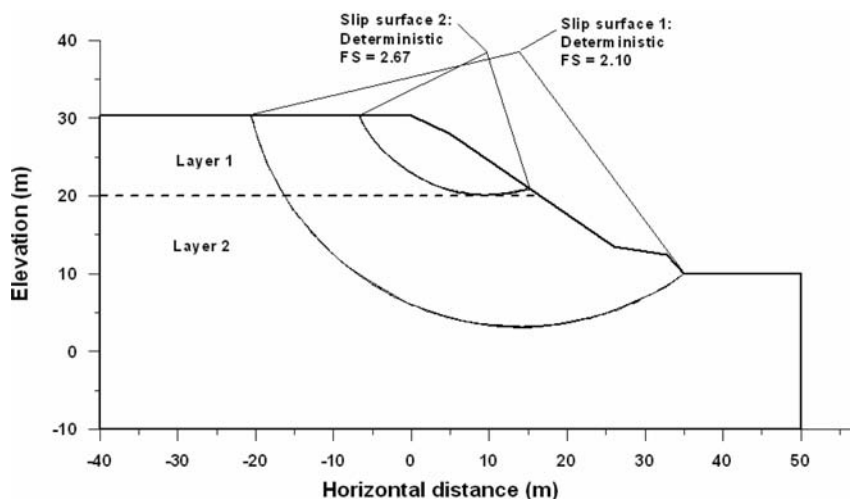


Figure 23. Reference slope for FORM analysis by Nadim & Lacasse (1999).

Table 16. Large-scale average property distributions for static and seismic conditions for FORM analysis (Nadim & Lacasse 1999).

Variable	Probability distribution	Mean	COV
s_u in Layer 1	Lognormal	75 kPa	25%
γ_t in Layer 1	Normal	17.5 kN/m ³	5%
s_u in Layer 2	Lognormal	150 kPa	15%
γ_t in Layer 2	Normal	17.5 kN/m ³	5%
Model uncertainty, ε	Normal	1.0	10%

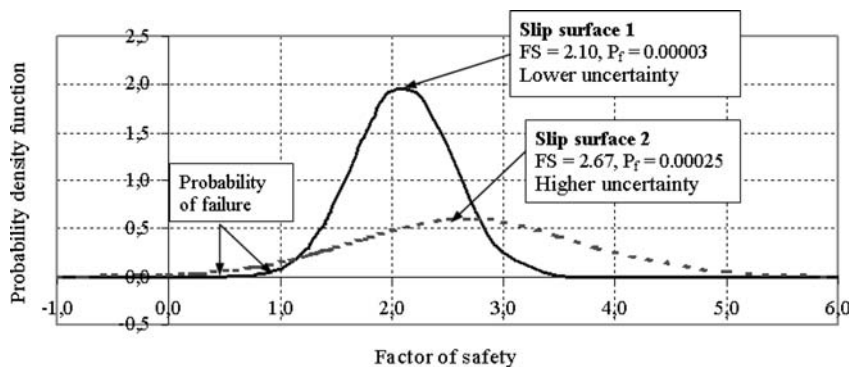


Figure 24. Computed distribution of safety factors for slip surfaces 1 and 2 (Nadim & Lacasse 1999).

statistics (i.e. mean and standard deviation) and statistical distributions, but could display substantial differences in spatial distribution. Figure 26 provides a comparison of two-dimensional spatial distribution of a generic parameter ξ having similar second-moment statistics and distributions (i.e. histograms), but different magnitudes of spatial correlation: weak correlation (top right) and strong correlation (bottom right).

Knowledge of the spatial behaviour of soil properties is often paramount in geotechnical analysis and design. Among the many reasons are: (a) geotechnical design is based on site characterisation, which objective is the description of the spatial variation of compositional and mechanical parameters of soils; (b) the values of the parameters themselves very often depend on in-situ state factors (e.g. stress level, overconsolidation ratio, etc.) which are related to spatial location; (c) for large-scale engineering

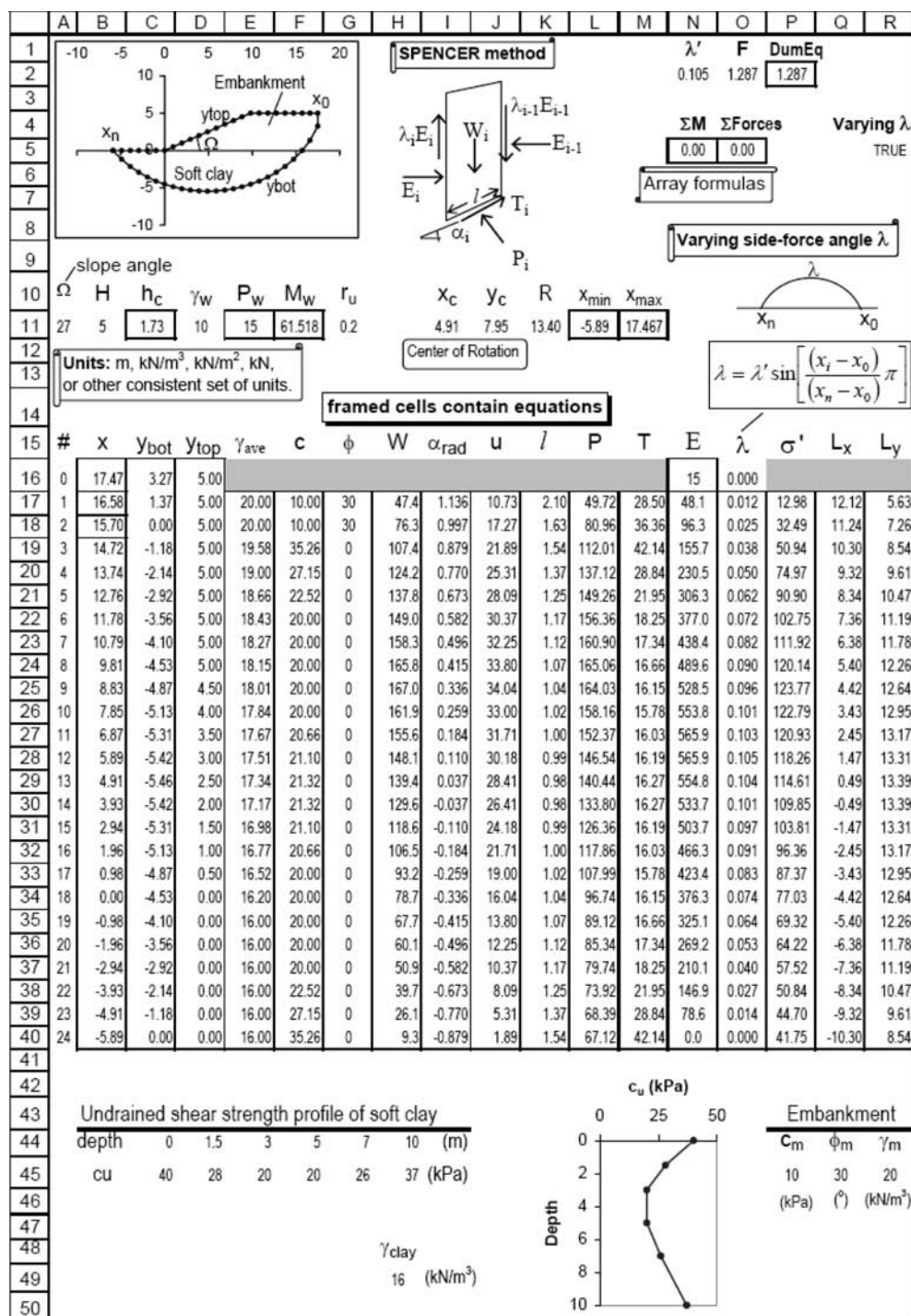


Figure 25. Spreadsheet search for reliability-based critical slip surface using the generalised method of slices (Low 2003).

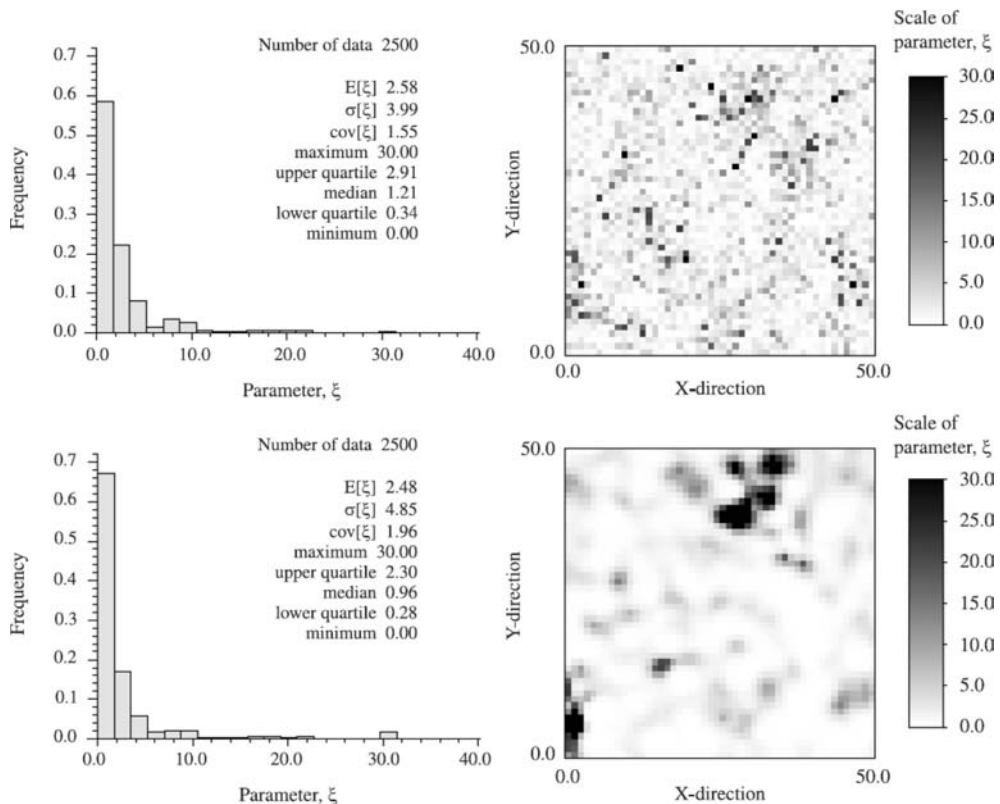


Figure 26. Comparative representation of spatial data with similar statistical distributions (top and bottom right) but different magnitudes of spatial correlation: weak correlation (top right) and strong correlation (bottom right) (from El-Ramly et al. 2002).

endeavours such as dams or roads, it is generally expected that heterogeneous site characteristics will be revealed by investigations at spatially distant locations.

3.1 Modelling spatial variability

The spatial variation of a soil deposit in any direction could, in principle, be characterised in detail if a sufficiently high number of measurements were taken. This, however, is impossible in practice, and could even be recognised as superfluous. Thus, as stated by Baecher (1982), the randomness in the variation of soil properties is often a *convenient* hypothesis to make as a result of this limitation in knowledge:

"There is clearly nothing random about the variation of soil properties from one location to another. In principle, with a sufficient number of measurements, the properties at every location could be known within the accuracy of testing. Practically, however, testing is limited. Thus, it is convenient to model the spatial variation of soil properties as if it were random, and to use associated statistical results to quantify interpolation errors. Actually, such an approach need never make assumptions of randomness. It need only recognize that the mathematics used to model random variations can also be used to summarize spatial behaviour, and then exploit that similarity to apply the mathematical results to spatial interpolation, averaging, or other problems."

Structures explanations of the statistical techniques used for the investigation spatial variability are provided by Priestley (1981) and Baecher & Christian (2003). While the former provides an exhaustive insight into the mathematical framework of time series analysis, the latter focuses specifically on application of such techniques to geotechnical engineering.

3.2 Stationarity and data transformation

Statistical modelling of spatial variability relies heavily on the hypothesis of data stationarity. While the implications and the methods to assess stationarity are addressed in some detail in Section 6, it may

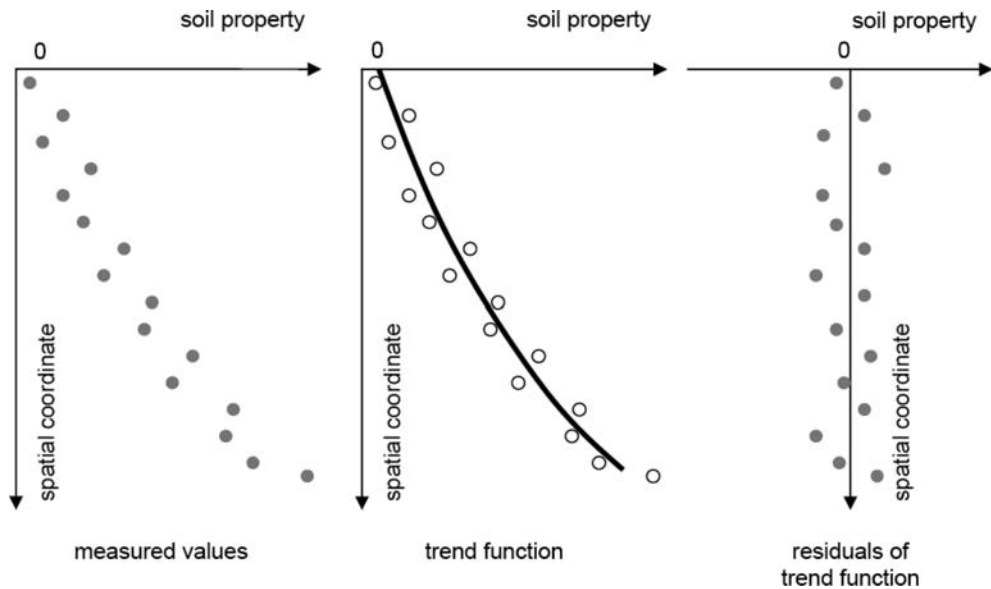


Figure 27. Visual representation of decomposition in the one-dimensional case.

be sufficient, here, to assert that stationarity denotes the invariance of a data set's statistics to spatial location. More specifically, *weak stationarity* is deemed sufficient to allow application of statistical techniques.

If a data set of interest is not stationary, the results of statistical analyses may be erroneous or biased. Hence, it is necessary to *transform* the data set. *Data transformation* is a general term referring to a number of techniques (mostly from time series analysis) which purpose is the transformation of a non-stationary data set to a stationary set. *Decomposition* is by far the most widely adopted data transformation technique in the geotechnical engineering literature. Such technique is illustrated in some detail in the following. Other data transformation techniques include *differencing* and *variance transformation*; a review of such methods is provided in Jaksa (2006).

3.3 Decomposition

By decomposition, the spatial variability of a spatially ordered measured geotechnical property [$\psi(z_1 \dots z_n)$] in a sufficiently physically homogeneous (according to some user-defined criterion) soil unit may be broken down into a *trend function* [$t(z_1 \dots z_n)$] and set of *residuals* about the trend [$\xi(z_1 \dots z_n)$]. In the one-dimensional case, for instance, taking depth (z) as the single spatial coordinate, decomposition is expressed by the following additive relation:

$$\psi(z) = t(z) + \xi(z) \quad (26)$$

An example is shown in Figure 27. Equation (26) neglects measurement error. This is justified by the hypothesis that the aleatory uncertainty resulting from inherent soil variability and the epistemic uncertainty due to measurement error are uncorrelated, and can be addressed separately. The presence of a spatial trend in soil properties, even for extremely homogeneous soils, is very common in geotechnical engineering, where the dependence of parameter magnitude from factors such as overburden stress and stress history is well recognised. The decomposition procedure, as spatial variability analyses in general, can be extended to higher-dimensional cases. Figure 28 (Przewłocki 2000) shows the decomposition of soil resistance data in a two-dimensional spatial extension. Hereinafter, reference will be made primarily to the one-dimensional case as this is most frequently encountered in practice and in the literature.

In the decomposition procedure outlined above, the trend is described deterministically by an equation; the residuals are characterised statistically as a variable, with (usually) zero-mean and (always) non-zero variance.

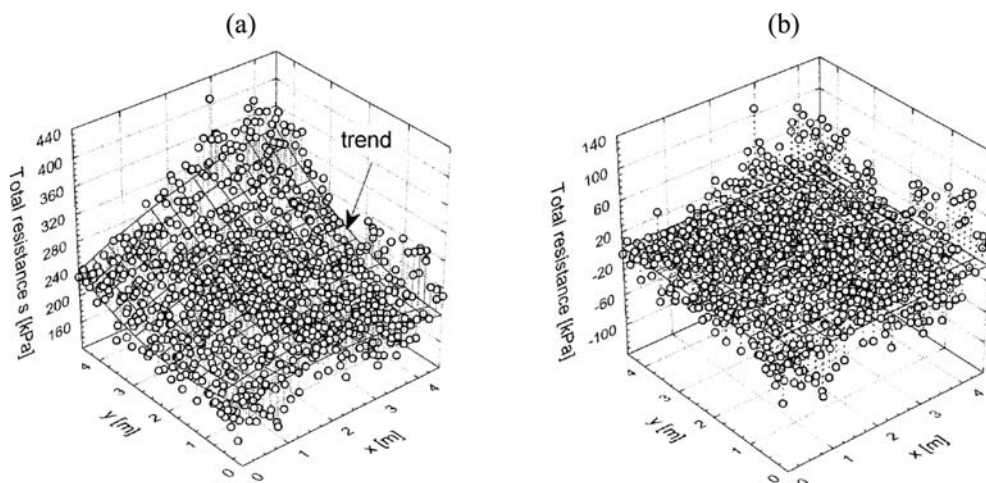


Figure 28. Example of 2-D decomposition of cone resistance: (a) measured data and trend surface; (b) residuals of detrending (Przewłocki 2000).

The decomposition procedure is arbitrary to some degree: first, the type of trend function (e.g. polynomial, exponential, etc.) to be used is established by the user; second, the estimation of trend parameters can be achieved using a variety of techniques.

The separation into a deterministic trend and random variation is thus an artefact of the analysis; there is no univocally “correct” trend to be identified, but rather a “most suitable” one. The choice of the trend function must be consistent with the requirements of the mathematical techniques adopted, and must, more importantly, rely on geotechnical expertise.

3.4 Spatial trend functions

The least-squares method is widely used to estimate the parameters to be fit to a set of data and to characterise the statistical properties of the estimates. Trend removal by least-squares regression has been utilised by several researchers in the geotechnical literature (e.g. Alonso & Krizek 1975; Baecher 1987; Campanella et al. 1987; Brockwell & Davis 1991; Kulhawy et al. 1992; Jaksa 1995; Jaksa et al. 1997; Phoon et al. 2003c; Wu 2003).

Regression analysis is useful in describing the relationship between two random variables (the soil property and the spatial direction variable). Such relationship is described in terms of the mean and variance of the first *conditioned* on the second. The main outputs of regression analysis are the *regression parameters* (i.e. the parameters describing the deterministic trend) and the *conditional variance* of soil property on the spatial coordinate. These account for, respectively, the assumed shape of the trend function and the part of total spatial variability which may be ascribed to the trend itself.

Figure 29 provides intuitive schemes of several possible regression cases, related to the type of relationship between variables (linear vs. non-linear) and the characteristics of data scatter around the trend with depth (constant vs. non-constant conditional variance). Trends are shown as solid lines; dispersion envelopes (e.g. trend $\pm$ conditional standard deviation) are shown as dashed lines.

Least-squares can be implemented in several versions: ordinary least-squares (OLS) relies on the hypothesis of homoscedasticity (i.e. constant variance of the residuals) and does not assign weights to data points; generalised least-squares (GLS) relaxes the hypothesis of homoscedasticity in favour of an independent estimate of the variance of the residuals, and allows for weighing of data points as a consequence of the variance model. Linear and non-linear OLS can be seen as special cases of GLS. Statistical regression is commonly performed using dedicated software. The reader is referred, for instance, to Ang & Tang (1975) for theoretical aspects of regression procedures.

3.4.1 Uncertainty in estimated trend parameters

Decomposition brings the effect of attributing some of the spatial variation of a soil property to the trend and some to the residuals. Even though the trend is a deterministic function, there is statistical estimation uncertainty due to the limited size of the data sets used for trend identification.

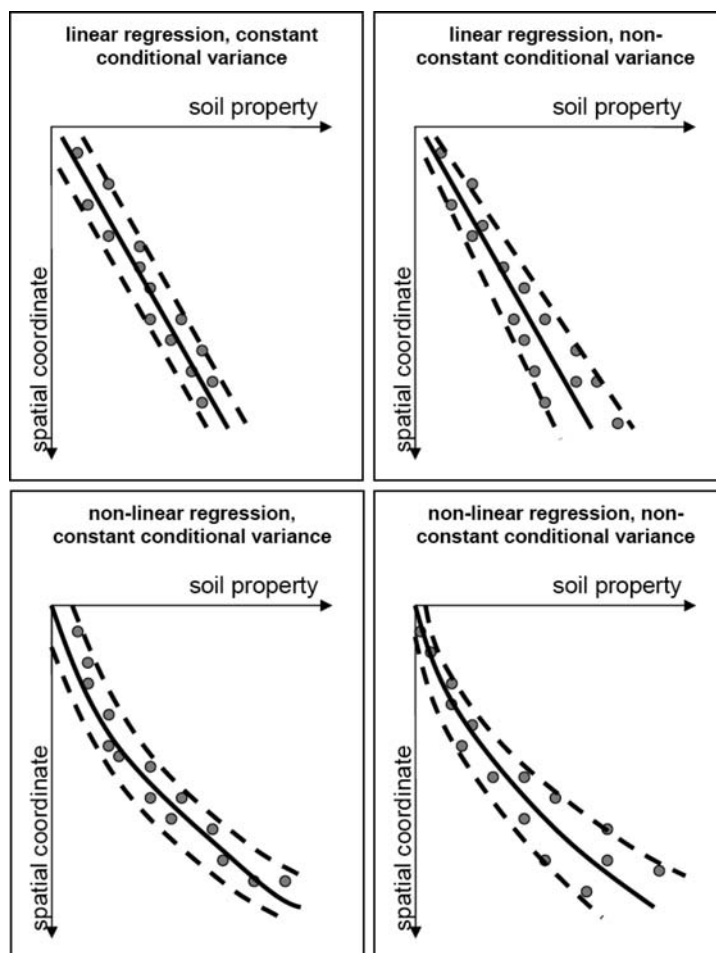


Figure 29. Possible cases of least-squares regression.

If a quantitative estimation of the spatial variability of soil properties is of interest, it is necessary to quantify the magnitude of the components of such variability, i.e. the variability related to the trend function and that pertaining to the set of residuals. Distinct approaches are required for the two components. Here, the estimation of the uncertainty in trend parameters is addressed. The uncertainty in the spatial variability of the set of residuals will be examined subsequently.

If the trend interpolates data values very closely, the variance of the residuals will decrease but the uncertainty in the estimation of trend parameters will be greater. Conversely, a simpler trend will have a smaller estimation uncertainty in its parameters, but the variance of the resulting set of residuals will be larger, approaching infinity as the trend interpolates the data perfectly. Figure 30 provides a comparative visual appreciation of the effects of removing a first- and a second-order polynomial from the same source data set. The results of first-order detrending still show a well-defined spatial trend (and are therefore unlikely to be stationary) and a larger variance than the residuals of second-order detrending, which show no appreciable spatial trend and which could be stationary (verification requires formal procedures, see Section 6).

3.4.2 Criteria for trend selection

While the formal compliance to the reference mathematical framework is necessary for the quantitative assessment of variability, the importance of geotechnical engineering judgement must not be downplayed because the presence of trends is usually motivated by geologic processes or other physical reasons which should be recognised by the geotechnical engineer. To avoid inconsistency with the underlying physical

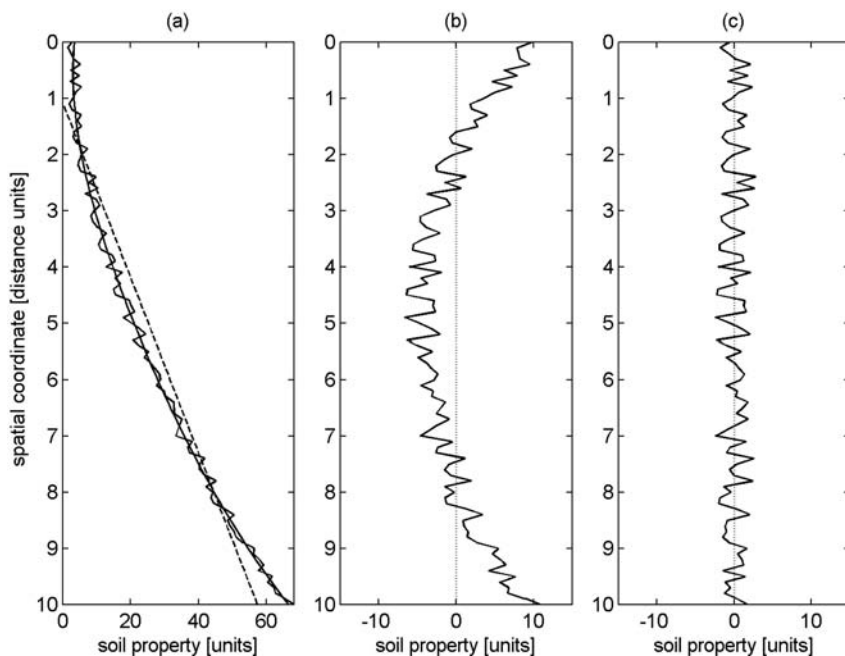


Figure 30. Comparative visual appreciation of the effects of removing a linear and a second-order polynomial from the same data set: (a) source data set with superimposed first- and second-order polynomials; (b) residuals of first-order detrending; (c) residuals of second-order detrending.

reality of the model under investigation, trends should not be removed if they are in contrast with – or are not amenable to – physical reasons (e.g. Akkaya & Vanmarcke 2003). In-situ overburden stress, for instance, is a major factor contributing to the occurrence of spatial trends in soil masses. Alonso & Krizek (1975) suggested that visual examination may provide the best means for trend identification. Baecher & Christian (2003) stated that “trends should be kept as simple as possible without doing injustice to a set of data or ignoring the geologic setting”.

Several numerical procedures for the assessment of the trend “quality” have been applied in the geotechnical literature: Cafaro & Cherubini (2002), for instance, appraised the appropriateness of removed trends by the CUSUM analysis technique (e.g. Cherubini et al. 2006). However, geotechnical expertise should play a central role in the selection of the complexity level of the trend. A study by Chiasson & Wang (2006) on the variability of sensitive Champlain marine clays provides an example of decomposition and trend analysis in a geotechnical perspective.

3.4.3 *Set-specificity of trend*

One of the main benefits of inferential statistical analyses is the possibility, if a reasonably conspicuous bulk of information is available from one or more sets, to make statements using about the stochastic nature of other presumably similar sets. The convenience of decomposition and the attribution of a part of the spatial variation to a deterministic trend have been discussed previously. It has been stated that the correlation structure of the residuals is different from that of the original data – it shows less spatial dependence and has reduced variance; the degree to which this occurs depends on the complexity of the trend, which is established by the user as noted previously. The use of only the residuals from a source site at the target site would lead to an unconservative underestimation of soil variability as the uncertainty pertaining to the trend would not be addressed.

A trend removed with reference to the minimisation of variance for one specific data set may be appropriate for one specific data set, but is generally not suitable for direct application to another set. It is thus conceptually incorrect to apply the trend obtained for a specific data set to other data sets (even if pertaining to the same geotechnical parameter) unless it has been verified that the criteria which were used for trend estimation are sufficiently similar in the target set. Trends which are qualitatively

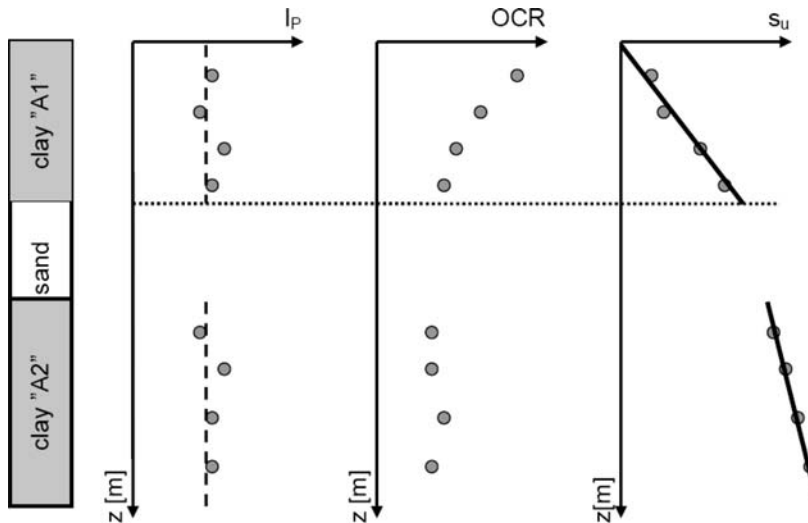


Figure 31. Example of set-specificity of spatial trend function.

associable to – and whose presumed associated level of uncertainty is similar to – the trend of the “source” should be expected at the target site.

Consider the example shown in Figure 31. Suppose that a boring log and a subsequent laboratory investigation have provided a stratigraphic profile indicating the presence of two clay layers “A1” and “A2” separated by a cohesionless layer. Atterberg limits measurements are performed on 4 samples for both A1 and A2, indicating that the plasticity index I_p is very similar for the two layers. Triaxial testing is performed on the samples as well, yielding the values of undrained shear strength shown on the figure. The overconsolidation ratio suggests that A1 is more heavily overconsolidated than A2. The trends identified for undrained shear strength s_u in A1 and A2 are very different. Despite the substantial homogeneity of A1 and A2 in index properties and the fact that linear trends are selected for both layers (and, hence, the magnitudes of the scatter of the residuals around the trends are presumably similar), it would be grossly incorrect to apply the trend of one layer to the other in the context of a spatial variability analysis.

In the context of soil variability modelling, non-exportability of trend functions is often referred to as “site-specificity” in the geotechnical literature; it is argued here that such terminology is not exhaustive and should be replaced by “set-specificity” because, as has been shown in the present example, distinct trends may occur even at the same site and, possibly, even in the same soil unit (consider, for instance, the variation of overconsolidation ratio with depth in highly homogeneous soils). Figure 32 illustrates the results of decomposition on submerged unit weight data for Troll marine clay (Uzielli et al. 2006). The difference in trends between the two physically homogeneous soil units (UN1 and UN2) is evident.

It is thus paramount to address spatial variability analysis with constant and careful reference to a geotechnical perspective; the latter should never be subordinated to mathematical convenience.

3.5 Residuals

The residuals of decomposition represent the part of spatial variability which cannot be explained by a relatively simple function of the reference spatial coordinate. They are usually a zero-mean set which, when plotted against the spatial coordinate, fluctuates around the mean value.

A fundamental assumption inherent to decomposition and trend fitting is that the residuals of decomposition are spatially uncorrelated, i.e. their fluctuation is completely random. This is equivalent to stating that the uncertainty in trend parameters accounts for the entire spatial structure of soil variability. As the level of complexity of the trend is chosen by the user, it would be possible to select a trend which would interpolate available data completely; however, the uncertainty in trend parameters would correspondingly approach infinity. It is more convenient to exploit the main advantage of decomposition, i.e. the possibility to identify a simple trend and to model the set of residuals as a random variable which may be effectively investigated using appropriate and well-established statistical techniques. In a

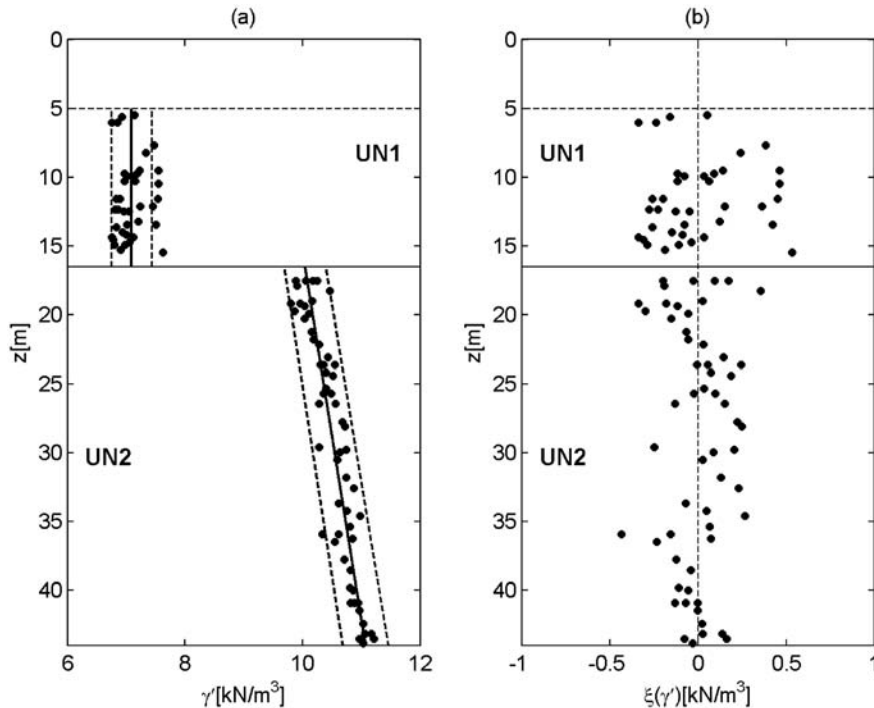


Figure 32. Decomposition and second-moment analysis for submerged unit weight of Troll marine clay: (a) original data, trends and standard deviation; (b) residuals (Uzielli et al. 2006).

second-moment perspective, the part of spatial variability that remains after trend removal corresponds to the variance of the residuals.

Investigation of the spatial pattern of the residuals is a central phase of variability modelling. Due to the natural processes which lead to the formation and modification of in-situ soil masses, soil properties vary more or less gradually along any spatial direction. Soil properties which are measured at contiguous spatial locations may well be expected to be closer, in magnitude, than properties measured at large spatial distances. Depending on the complexity of the removed trend, this is also true for the set of residuals. The more complex the trend, the less the spatial structure of residuals is preserved.

The spatial pattern which remains in the values of soil properties after the removal of a deterministic trend can be referred to as a *spatial correlation structure* or, equivalently, *autocorrelation*, as it refers to correlation between elements of the same set.

3.5.1 Autocorrelation and variance of residuals

An insight into the meaning of correlation and variance is warranted. Figure 33 illustrates qualitatively the meaning of “weak” or “strong” spatial correlation and “low” and “high” variance. The horizontal and vertical axes of the four plots of hypothetical residuals of decomposition have the same horizontal and vertical scales. In most intuitive terms, the magnitude of spatial correlation is shown by the “roughness” of the plot, with weak spatial correlation attesting for a “rougher” and less undulated plot than strong spatial correlation. The magnitude of variance is reflected in the amplitude of the oscillations, with a higher variance resulting in greater amplitude. In principle, spatial correlation and variance are not related, i.e. the four cases are equally plausible *a priori*.

The decomposition procedure is to some degree arbitrary, in the sense the user should identify a suitable trend function based on geotechnical considerations and in compliance with the requirements of the statistical methods employed to quantify variability. As a trend is associated with exactly one set of residuals, the latter are also (though indirectly) somewhat user-determined. The subjectivity lies in the way in which autocorrelation is modelled, and not on the real spatial behaviour of a soil property which is modelled as a random variable due to the user’s insufficient knowledge.

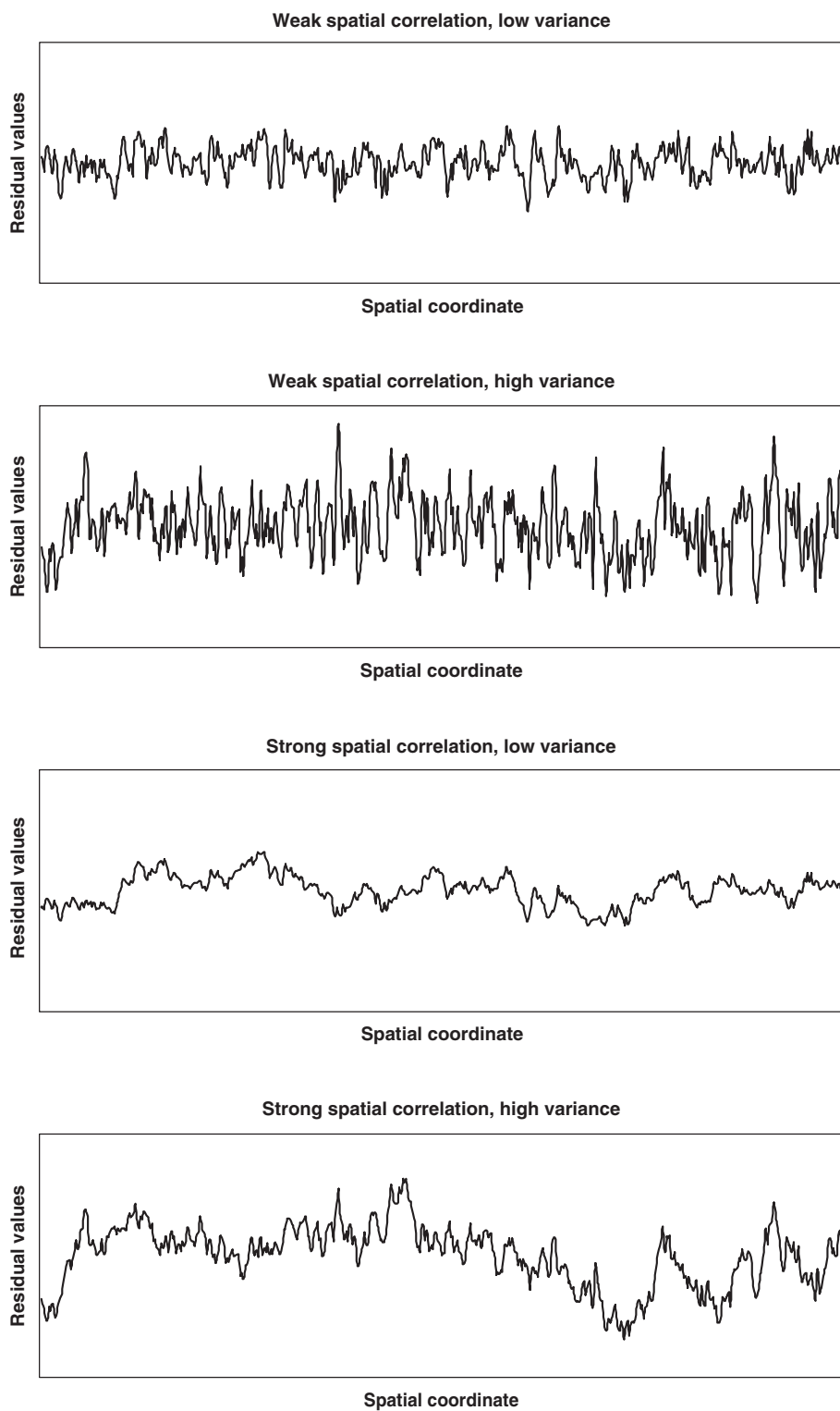


Figure 33. Implications of spatial correlation and variance.

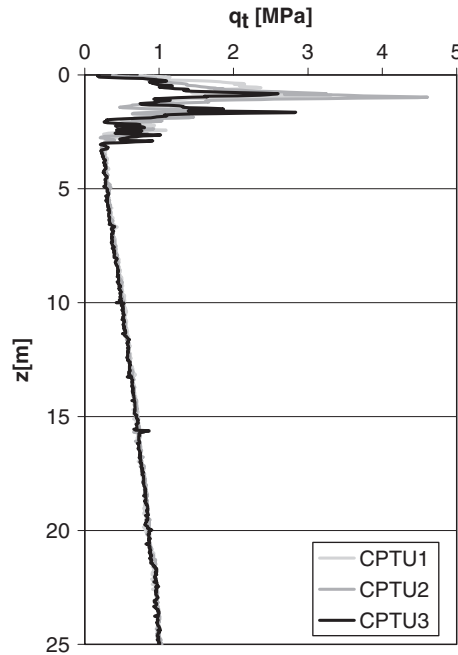


Figure 34. Superimposed plots of corrected cone tip resistance profiles from Onsoy CPTU data.

3.5.2 Anisotropy in spatial correlation

The way in which soil properties vary along different spatial directions is generally different. This is to be expected, given the characteristics of depositional and soil modification processes. Spatial variability is, in general, markedly *anisotropic*, with a larger degree of homogeneity (and, hence, a stronger correlation structure) in the horizontal direction. This is due to mainly to the fact that most depositional processes result in stratigraphic geometries which are usually not significantly distant from a set of horizontal layers whose vertical thickness is much smaller than the horizontal extension. The degree of anisotropy depends on soil type as well as in-situ conditions.

As an example, Figure 34 shows three superimposed profiles of corrected cone tip resistance q_t (e.g. Lunne et al. 1997) for Onsoy clay in Norway. Based on visual inspection, the presence of two layers (in terms of mechanical behaviour) is evident, as well as the following: (a) the vertical autocorrelation of the bottom layer is stronger than that of the top layer (the dispersion around a quasi-linear trend is very small in the bottom layer, while the data scatter around any reasonably simple trend would be considerable for the top layer); and (b) the horizontal autocorrelation is very strong for the bottom layer (the profiles from different spatial locations are extremely similar), while this is less marked for the top layer even though there are substantial similarities in the 3 profiles.

Anisotropy should be recognised in each phase of spatial variability estimation, starting from the decomposition procedure (different trends should be identified for different spatial directions). It is paramount to state clearly which spatial direction(s) the results of a spatial variability analysis refers to. This will be shown in Section 4.

3.5.3 Distribution of residuals

The assumption of Gaussianity of residuals is strictly not necessary for the estimation of the variance of the residuals themselves. However, it is usually an acceptable hypothesis in geotechnical spatial variability analyses, and is convenient for the application of a number of procedures.

3.5.4 Stationarity of residuals

The main reason for performing data transformation (e.g. by decomposition) is to obtain *stationary* residuals. This is desirable because virtually all classical statistics are based on the assumption of stationarity. Stationarity will be addressed in greater detail in Section 6.

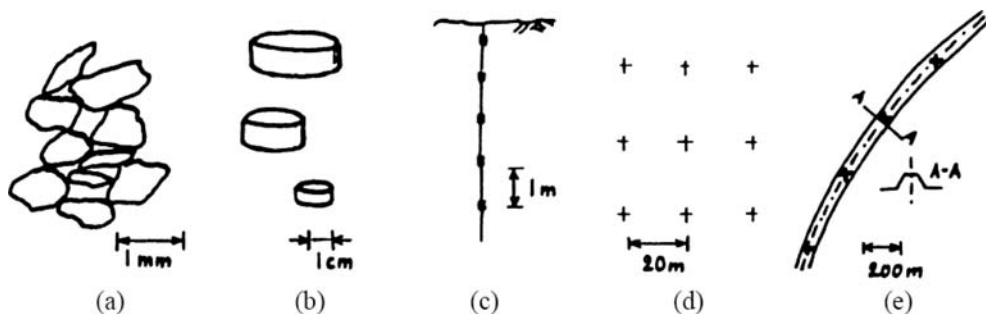


Figure 35. Scales of spatial variability modelling in geotechnical engineering (Vanmarcke 1978).

3.5.5 Fractal vs. finite-scale approaches to autocorrelation estimation

It has been stated earlier that the natural processes which result in the formation and modification of soil masses induce a marked anisotropy in the spatial distribution of most soil properties, with the horizontal correlation generally being stronger. However, it is intuitive that the effects of natural processes may occur, both at large and smaller scales. Hence, it should be expected – for a given soil parameter of interest – that correlation structures presenting heterogeneous spatial magnitudes may be simultaneously present.

Figure 35 (Vanmarcke 1978) illustrates the multiplicity of scales of investigation which are possible in geotechnical engineering: (a) soil particles; (b) laboratory specimen; (c) vertical sampling; (d) lateral distances between borings; (e) horizontal intervals at regional scale, e.g. measured along the centreline of long linear facilities. Distinct measurement techniques are generally employed for the various scales. This suggests that the magnitude of spatial correlation also depends from the method and scale of investigation.

The dependence of the correlation structure on the scale of investigation has led a number of researchers (e.g. Agterberg 1970; Campanella et al. 1987; Fenton 1999a, 1999b; Fenton & Vanmarcke 2003; Jaksa 1995; Kulatilake & Um 2003) to address autocorrelation in soils using *fractal models*, which assume an infinite correlation structure and allow to directly address the dependence of the correlation structure from the sampling domain.

Figure 36 (Fenton 1999a) provides a visual representation of a Gaussian fractal process seen at various resolutions. It may be appreciated that the profile is self-similar at every scale. For such a process, quantitative estimates of spatial correlation would depend on the scale at which the process is observed.

Assessment of the fractal nature of soil properties, though using a more geotechnical than mathematical terminology, is common in the geotechnical engineering literature. The geotechnical engineer is well accustomed to investigating soil properties at different spatial scales. Simonini et al. (2006), for instance, describe the soils of the Venice lagoon as follows: “[...] the majority of soils belong to sands, silts and silty clays with the presence of several layers of peat: the high heterogeneity is clearly evident at a larger scale, but significant material variation may be observed even at centimetre scale.” Figure 37 provides information regarding the large-scale vertical spatial variability of a number of soil properties from the Malamocco site.

Vanmarcke (1983) observed that it is a matter of pragmatism to recognise that, in all practical applications, there invariably exists a spatial scale below which “micro-scale” or “macro-scale” (in relative terms) variations are: (a) not observed or observable; and (b) of no practical interest to the situation at hand. This observation is certainly pertinent in geotechnical engineering. As an example, Figure 38 shows the results of an investigation on the vertical spatial heterogeneity of a soil sample from the Malamocco Venice lagoon soils investigated by Simonini et al. (2006): unit weight and void ratio vary significantly within the sample. Lower-scale boundary is imposed by the resolution of measurement instrumentation and the upper-scale boundary is given by the dimensions of the sample.

Such pragmatic considerations justify a different approach which consists in assuming that the spatial correlation of any soil property has a lower and upper bound, and is independent from the scale of investigation. In this case, autocorrelation is modelled as a *finite-scale* stochastic process.

From the point of view of numerical estimation, while the aforementioned dedicated studies have confirmed the theoretical appropriateness of the assumption of fractal behaviour of soil properties, practical applications (e.g. Jaksa & Fenton 2002) have shown that the vast majority of geotechnical data

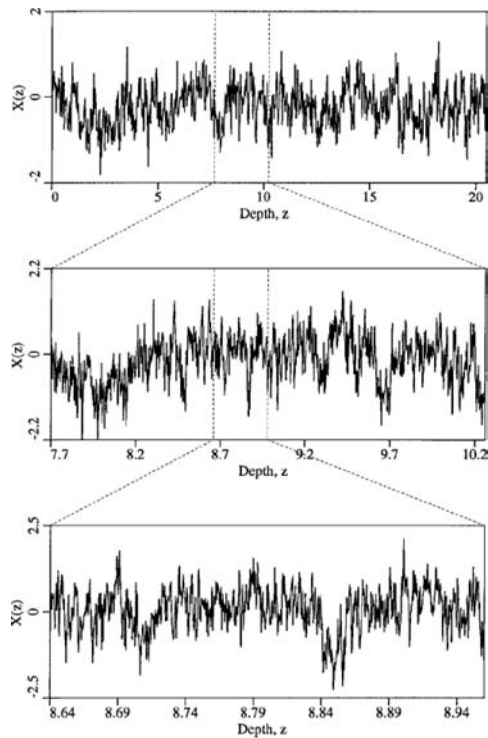


Figure 36. Visual representation of a Gaussian fractal process seen at various resolutions (Fenton 1999a).

can be modelled using a finite-scale approach. Moreover, it has been suggested (e.g. Fenton 1999a; Phoon et al. 2003c) that: (a) it is difficult to distinguish between finite-scale and fractal models over a finite sampling domain; (b) once a reference spatial extension has been established, there may be little difference between a properly selected finite-scale model and the real fractal model over the finite domain.

While the use of simulation techniques could provide an effective means of establishing the relationship between such an “effective” finite-scale model and the “true” but finite-domain fractal model, the present state of knowledge would only allow, for practical application purposes, to refer to finite-scale models. Hence, fractal models will not be addressed further in the present paper.

3.5.6 Finite-scale autocorrelation estimation approaches

Available finite-scale approaches for quantifying autocorrelation essentially include Bayesian and frequentist approaches. Baecher & Christian (2003) provided an insight into the Bayesian approach to autocorrelation estimation. Such approach has not been widely used in the geosciences literature, and will not be addressed further in the present paper.

Common frequentist approaches include *maximum likelihood estimation* and *moment estimation*. Several researchers (e.g. Fenton 1999a; Baecher & Christian 2003) have advocated the use of maximum likelihood estimation over moment estimation, as the latter are more prone to bias in case of long-scale correlation. Also, maximum likelihood allows simultaneous estimation of the spatial trend and the autocorrelation in residuals. While a number of contributions addressing maximum likelihood estimation are available in the geotechnical literature (e.g. DeGroot & Baecher 1993; DeGroot 1996; Fenton 1999a; Baecher & Christian 2003), most of the currently available data regarding the variability of geotechnical parameters was obtained by moment estimation techniques.

Moment estimators use the statistical moments of a set of data (e.g. means, variances, correlation) as estimators of the corresponding populations which are being sampled, and whose real moments are not known. Moment estimators are conceptually and operationally simple, and have the advantage of being non-parametric (i.e. they do not require knowledge or assumption regarding population distributions, but only require that the moments of the distributions may be estimated). However, they may not be able

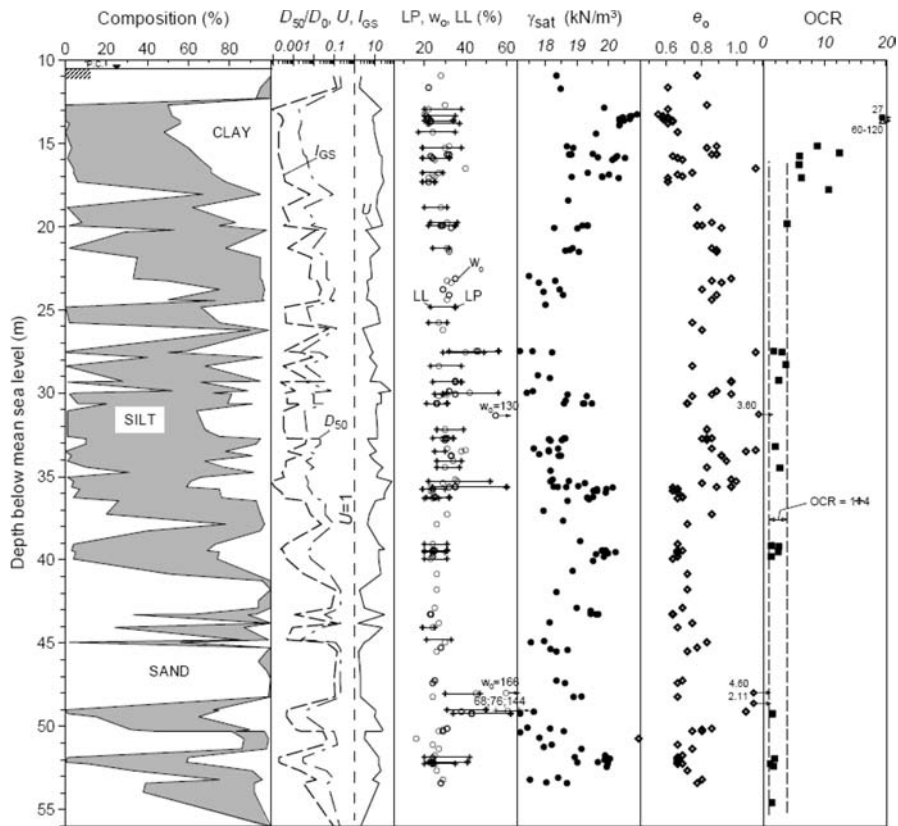


Figure 37. Results of geotechnical testing at Malamocco site in the Venice lagoon showing large-scale variability (Simonini et al. 2006).

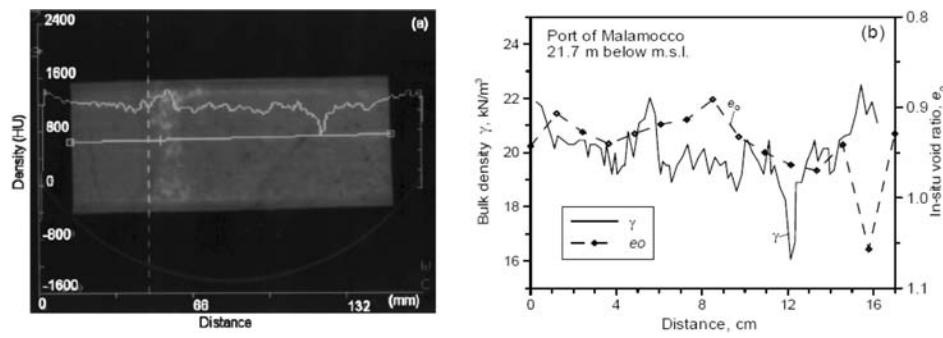


Figure 38. Tomographic analysis on a soil sample from the Malamocco test site in the Venice lagoon: (a) screenshot of density in Hounsfield units; (b) bulk density in SI units and in-situ void ratio (Simonini et al. 2006).

to represent the populations sufficiently well, especially for small sample sizes, as sample moments are always biased in practice.

It is well known that the sample autocorrelation function estimated from data is “noisy” at large lags. The presence of noise complicates the interpretation of how the actual autocorrelation function decays with lag distance and is clearly undesirable. However, it is rarely appreciated that the fluctuations at large lags which reduce the quality of interpretation are expected *theoretically*. More precisely, the sample autocorrelation function is a non-stationary stochastic process with mean and variance that

are functions of the lag distance. A more hopeful strategy is to exploit growing desktop computational power and increasingly powerful simulation techniques for stochastic processes to perform bootstrapping. Numerical results presented by Phoon & Fenton (2004) and Phoon (2006a) show that fairly accurate mean estimates of the sample autocorrelation function can be obtained for both Gaussian and non-Gaussian processes using bootstrapping. Variance estimates are less accurate, but even crude estimates are useful in identifying the level of noise at large lags and reducing misinterpretation of how the actual autocorrelation function decays with lag distance.

3.5.6.1 Autocorrelation function

The autocorrelation function of a stochastic process (which must be at least weakly stationary) describes the variation of the strength of spatial correlation as a function of the spatial separation distance between two spatial locations at which data are available.

As stated previously, it is not possible to calculate the real autocorrelation function of a stochastic process (and, thus, to investigate its *true* spatial correlation structure) due to the fact that data sets are, in practice, always limited in size. Hence, it is necessary to refer to the *sample autocorrelation function*, i.e. to an approximation of the real autocorrelation function, calculated from an available set of data which is deemed representative of the stochastic process.

In the discrete case, the sample autocorrelation sample of a set of data is given by:

$$\hat{R}(\tau_j) = \frac{\sum_{i=1}^{n-|j|} (\xi_i - m_\xi)(\xi_{i+|j|} - m_\xi)}{\sum_{i=1}^n (\xi_i - m_\xi)^2} \quad (27)$$

in which m_ξ is the mean of the data set. The function should be estimated for separation distances $|\tau_j| = j\Delta z$ corresponding to $j = 1, 2, \dots, n/4$, as suggested by Box & Jenkins (1970), where n is the number of data points and Δz is the (constant) sampling interval.

It should be noted that absolute values have been used to denote separation distance. This is due to the fact that the autocorrelation function is symmetric about zero separation distance (essentially represents the fact that autocorrelation can be investigated in both directions from a given spatial location). However, due to the aforementioned symmetry, only the positive separation distance domain of autocorrelation functions is usually represented graphically.

Unless specific techniques such as the Galerkin polynomial trend fitting method are used, decomposition results in a zero-mean set of residuals. In the case of zero-mean sets, Eq. (27) can be rewritten as:

$$\hat{R}(\tau_j) = \frac{\sum_{i=1}^{n-|j|} \xi_i \cdot \xi_{i+|j|}}{\sum_{i=1}^{n-|j|} \xi_i^2} \quad (28)$$

The autocorrelation at zero separation distance is equal to 1.0, which is its maximum value; empirically, for most geotechnical data, autocorrelation tends to zero with increasing separation distance, and can assume negative values.

Figure 39 provides a visual comparison between two sample autocorrelation functions: ACF1, calculated from a data set with a weaker spatial correlation structure, decreases more rapidly to zero than ACF2, representing a data set with stronger correlation structure. A data set with no spatial correlation (i.e. white noise) has unit autocorrelation at zero separation distance and zero autocorrelation otherwise.

Fenton (1999a) showed that the sample autocorrelation function is useful in the estimation of spatial correlation if such correlation is considerably smaller than the extension of the sampling domain. If, on the contrary, the process has long-scale dependence, the sample autocorrelation function is unable to detect this, and continues to illustrate short-scale behaviour.

The autocorrelation function has been widely used for investigating spatial variability in the context of geotechnical engineering (e.g. Akkaya & Vanmarcke 2003; Baecher 1982, 1986, Baecher & Christian

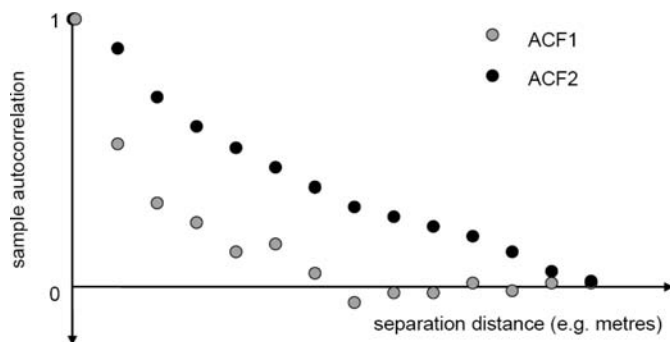


Figure 39. Examples of autocorrelation functions with weaker (ACF1) and stronger (ACF2) spatial correlation.

Table 17. Selected theoretical autocorrelation models as a function of separation distance τ .

Autocorrelation model	Formula
Single exponential (SNX)	$R(\tau) = \exp(-k_{SNX} \tau)$
Cosine exponential (CSX)	$R(\tau) = \exp(-k_{CSX} \tau) \cos(k_{CSX}\tau)$
Second-order Markov (SMK)	$R(\tau) = (1 + k_{SMK} \tau) \exp(-k_{SMK} \tau)$
Squared exponential (SQX)	$R(\tau) = \exp[-(k_{SQX}\tau)^2]$

2003; Cafaro et al. 2000; Cafaro & Cherubini 2002; Campanella et al. 1987; Christian et al. 1994; DeGroot & Baecher 1993; DeGroot 1996; Fenton 1999b; Fenton & Vanmarcke 2003; Jaksa 1995; Jaksa et al. 1997, 2000; Phoon & Fenton (2004); Phoon & Kulhawy 1996, 1999a, 1999b; Phoon et al. 2003c, 2004; Uzielli 2004; Uzielli et al. 2005a, 2005b; Vanmarcke 1983; White 1993; Wickremesinghe & Campanella 1993; Wu & El-Jandali 1985; Wu 2003).

To identify *how* a given soil properties varies spatially, one needs to fit one or more *autocorrelation models* to the sample autocorrelation function calculated by (28). An autocorrelation model quantifies autocorrelation as a function of separation distance, and is described by a function and a characteristic model parameter. Various autocorrelation models have been employed in the geotechnical literature (e.g. Spry et al. 1988; DeGroot & Baecher 1993; Jaksa 1995; Lacasse & Nadim 1996; Fenton 1999b; Phoon et al. 2003c; Uzielli et al. 2005a, 2005b). The equations relating the autocorrelation function, $R(\tau)$, and separation distance (τ) for four models are given in Table 17.

According to Spry et al. (1988), no autocorrelation model is univocally preferable over others on the basis of physical motivations. Figure 40 (Uzielli et al. 2005a) shows, for instance, the distribution of best-fit autocorrelation models for soil units which were found to be physically homogeneous in terms of stress-normalised cone tip resistance. Each point on the CPT-based soil behaviour classification chart by Robertson (1990) represents the mean values of normalised friction ratio F_R (Wroth 1984) and normalised cone tip resistance q_{c1N} (Robertson & Wride 1998) in a physically homogeneous soil unit. It may be observed that no relation exists between best-fit autocorrelation models and soil type (cohesive soils in zones 2 and 3; intermediate behaviour soils in zone 4 and cohesionless soils in zones 5 and 6), as well as data points representative of profiles from physically homogeneous soil units for which weak stationarity could not be investigated in a rigorous way using the MBSR method (see §6.1.2) because of insufficiently reliable estimates of the scale of fluctuation (NAPP) or because they were classified as non-stationary (NST). Future research could contribute to a clarification of the issue.

As there does not appear to be a reliable physical correspondence between soil type and the type of correlation structure, a purely numerical approach is justified. The choice of the correlation structure for a given data set can be based, for instance, on the comparative assessment of goodness-of-fit of one or more theoretical autocorrelation models to the empirical sample autocorrelation function of the data set itself. The procedure requires the optimisation of the characteristic model parameter for each autocorrelation model (e.g. by least-squares regression or other optimisation procedures) and the subsequent calculation of the goodness-of-fit parameter. The autocorrelation model yielding the maximum determination coefficient could be selected as best-fit model.

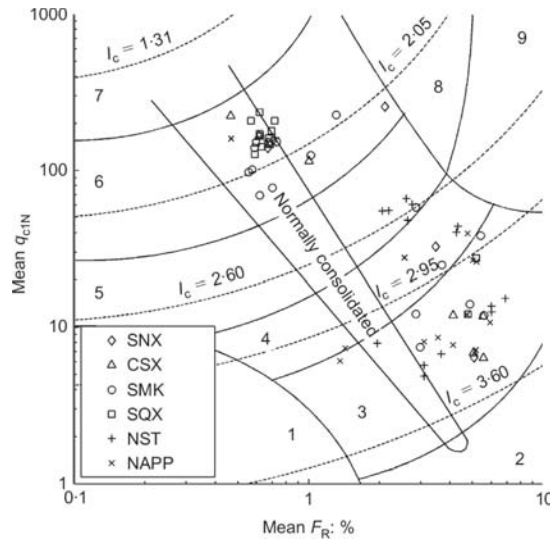


Figure 40. Distribution of best-fit autocorrelation models for stress-normalised cone tip resistance in physically homogeneous soil units (Uzielli et al. 2005a).

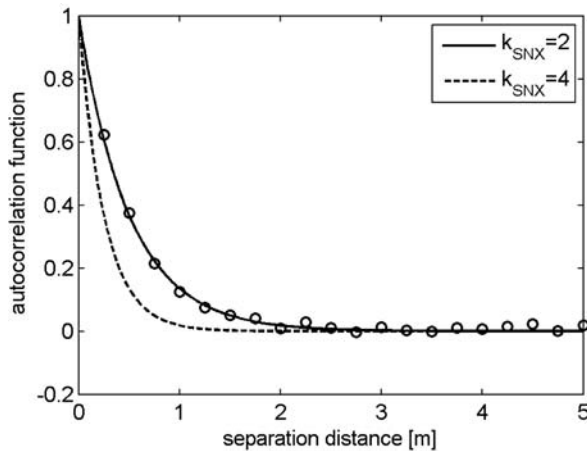


Figure 41. Fit of single exponential (SNX) model curves with different characteristic model parameters to an empirically calculated sample autocorrelation function.

Figure 41 illustrates the meaning of optimisation for a specific autocorrelation model. Two curves from the single exponential model (one with $k_{SNX} = 2$ and $k_{SNX} = 4$) are fitted to a sample autocorrelation function. It can be observed that the curve with $k_{SNX} = 2$ provides a significantly better fit.

Figure 42 shows the importance of identifying the most suitable autocorrelation model. In the figure, the optimised single exponential and squared exponential models are fitted to a sample autocorrelation function. It may be seen that the single exponential model approximates the correlation structure of the data set better than the squared exponential.

Figure 43 illustrates a real-case application of the autocorrelation model fitting explained above for data from a CPTU sounding performed in Brindisi, Italy (Cherubini et al. 2006). The sample autocorrelation function is shown along with four theoretical autocorrelation models which characteristic model parameters have been optimised. The single exponential model with $k_{SNX} = 1.23$ provides the best fit.

With the aim of setting stricter criteria on the choice of the correlation structure of the residuals, Spry et al. (1988) opined that the autocorrelation structure should be identified based only to the initial part of

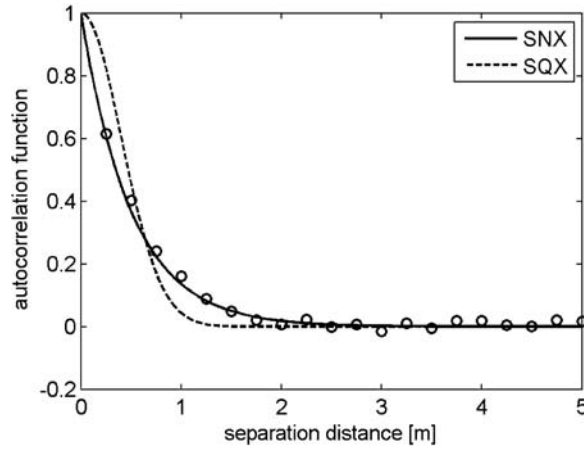


Figure 42. Curve fit of single exponential (SNX) and squared exponential (SQX) models to an empirically calculated sample autocorrelation function.

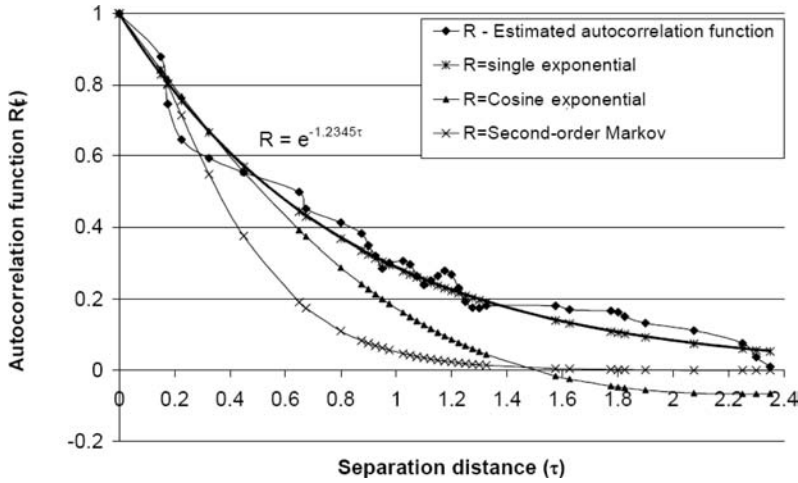


Figure 43. Sample autocorrelation function and autocorrelation model fitting for a soil from the Brindisi area (Cherubini et al. 2006).

the sample autocorrelation function, i.e. by fitting empirical autocorrelation models to the autocorrelation coefficients exceeding *Bartlett's limits*, which can be approximated for practical purposes by:

$$l_B = \frac{1.96}{\sqrt{n_d}} \quad (29)$$

This restrictive condition is motivated by the fact that the estimated autocorrelation coefficients become less reliable with increasing separation distance, and are deemed not significantly different from zero inside the range $\pm l_B$ (e.g. Priestley 1981; Fenton 1999a).

With the aim of establishing yet more restrictive criteria for the acceptance of the correlation structure, Uzielli (2004) suggested that a minimum R^2 value be set for the autocorrelation model fit to the sample autocorrelation function. Moreover, to ensure that the fit is significant, Uzielli (2004) opined that at least 4 autocorrelation coefficients should be used to fit the autocorrelation model and to calculate the fit parameter R^2 .

Table 18. Selected theoretical semivariogram models as a function of separation distance τ .

Semivariogram model	Formula
Spherical (SPH)	$V(\tau) = C \left(\frac{3 \tau }{2a_{SPH}} - \frac{ \tau ^3}{2a_{SPH}^3} \right) + C_0 \quad \text{for } \tau \leq a_{SPH}$ $V(\tau) = C + C_0 \quad \text{for } \tau > a_{SPH}$
Exponential (EXP)	$V(\tau) = C \left[1 - \exp \left(-\frac{ \tau }{a_{EXP}} \right) \right] + C_0$
Gaussian (GAU)	$V(\tau) = C \left[1 - \exp \left(-\frac{ \tau ^2}{a_{EXP}^2} \right) \right] + C_0$

3.5.6.2 Semivariogram

The semivariogram provides essentially the same information as the autocorrelation function. However, it has the advantage of being an unbiased estimator as it does not depend on the mean of the data set. As for the autocorrelation function, the exact shape of the semivariogram cannot be known from a sample of limited size. Hence, it is necessary to estimate the *sample semivariogram* of a data set of n elements:

$$\hat{V}(\tau_j) = \frac{1}{2(n-j)} \sum_{i=1}^{n-j} (\xi_{i+j} - \xi_i)^2 \quad (30)$$

(for $j = 0, 1, \dots, n-1$). The sample semivariogram may be used to investigate the spatial correlation structure of a specific data set.

The semivariogram requires a less restrictive statistical assumption regarding stationarity than the autocovariance function, as it does not require stationarity of the mean (in other words, trends in data can be accepted). However, it has often been recommended (e.g. Journel and Huijbregts 1978) that data should be detrended. Theoretical semivariogram models are available; a number of these is illustrated in Table 18.

The parameter C_0 is referred to as the *nugget*, and represents small-scale variability; the parameter a is termed *range*, and describes the maximum distance at which correlation between observations may be assumed; $C + C_0$ is the *sill*.

All of the above models obey the condition $V(0) = 0$. However, C_0 generally assumes non-zero values, and it appears that the semivariogram has a non-zero intercept. This is representative of the *nugget effect*, and attests for the existence of a lower-bound spatial distance in which the random process of interest is so erratic that the semivariogram goes from zero to C_0 in a distance less than the sampling interval. The nugget effect (addressed in greater detail in Jaksa 1995) is due essentially to small-scale variability as well as measurement error. The two components are indistinguishable in practice. A discussion on small-scale variability is provided in §3.7. Figure 44 provides a comparative plot of the spherical, exponential and gaussian theoretical semivariogram models, with range equal to unity ($a_{SPH} = a_{EXP} = a_{GAU} = 1$) and $C_0 = 1$.

Investigation of the correlation structure by means of the semivariogram is conceptually analogous to the procedure based on the sample autocorrelation function. Optimisation for one or more semivariogram models is followed by the selection of the best-fit semivariogram model. Journel and Huijbregts (1978) suggested that automatic fitting of models to experimental semivariograms (e.g. by least-squares) should be avoided because each point of an empirical semivariogram is calculated using a different number of data points (as a function of distance); hence, the estimation error is not uniform. Hence, a weighted fit (with weighting function based on critical appraisal of data and on experience) is preferable. In the case of the autocorrelation function, Bartlett's limits can be adopted as a threshold for determining which autocorrelation coefficients are sufficiently reliable; no equivalent parameters are available for experimental semivariograms. Hence, the use of the latter is somewhat more uncertain in comparison with the former. Figure 45 (Jaksa 1995) provides visual examples of: (a) excellent fit; and (b) poor fit of the spherical model to sample semivariograms.

The semivariogram has frequently been used in the geotechnical literature (e.g. Azzouz et al. 1987; Azzouz & Bacconnet 1991; Baecher 1984; Brooker et al. 1995; Chiasson et al. 1995; DeGroot 1996; Elkateb et al. 2003a, 2003b; Fenton 1999a; Hegazy et al. 1996; Jaksa 1995; Jaksa et al. 1993; Krige 1951; Kulatilake & Miller 1987; Kulatilake & Ghosh 1988; Kulatilake & Um 2003; Matheron 1963; Nobre & Sykes 1992; Nowatzky et al. 1989; O'Neill & Yoon 2003; Oullet et al. 1987; Soulié 1983; Soulié et al. 1990; White 1993).

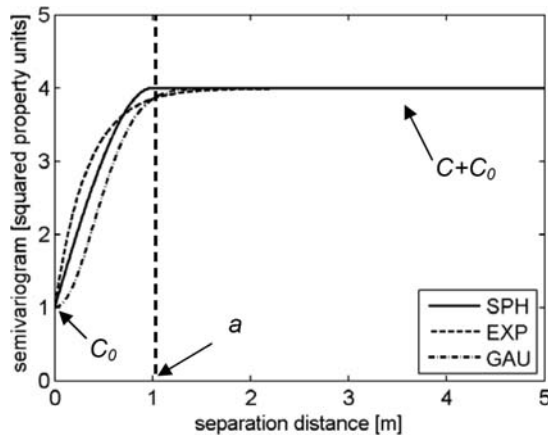


Figure 44. Plot of semivariogram models for range equal to unity and $C_0 = 1$.

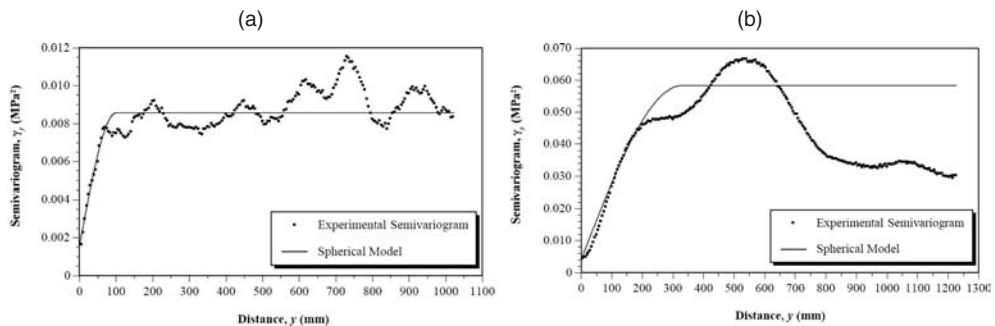


Figure 45. Example fits of sample semivariograms to spherical semivariogram model: (a) excellent fit; and (b) poor fit (Jaksa 1995).

3.6 Quality of data for autocorrelation analysis

In the second-moment perspective, sample numerosity and outlier exclusion were sufficient criteria for assessing the quality of a data set (statistical estimation error is reduced). In addressing the spatial variability of data and autocorrelation, additional criteria are required. For instance, the techniques used to model soil variability are greatly simplified if the measurement interval in data sets is constant. Alonso & Krizek (1975) opined that a good definition of soil variability requires that continuous or quasi-continuous records of soil properties be used “to show both the random character of the soil mass and the correlation of properties from point to point”.

For non-uniformly spaced data, the sample autocorrelation function can still be used, but should, in strict terms, require dividing separation distances into bands, and then taking the averages of autocorrelation within those bands (Baecher & Christian 2003). In practice, for testing methods with frequent recordings such as the CPT, measurement depths can be rounded to regular interval series and assuming strong property autocorrelation within the reference interval without introducing significant errors. Jaksa (1995) referred to this procedure as *rationalisation*. For high-interval tests such as the SPT it is harder to perform rationalisation as the hypothesis of strong correlation over significantly longer measurement intervals is much less acceptable.

3.7 Small-scale variability

It has been stated previously that a scatter due to small-scale but real variations in a measured soil property is at times mistakenly attributed to measurement error. In practice, it is extremely difficult to separate such sources of uncertainty.

It should be noted, however, that laboratory and in-situ geotechnical measurements are based on the generation of some type of failure and plastic deformation of a spatial volume of soil. Thus, measurements

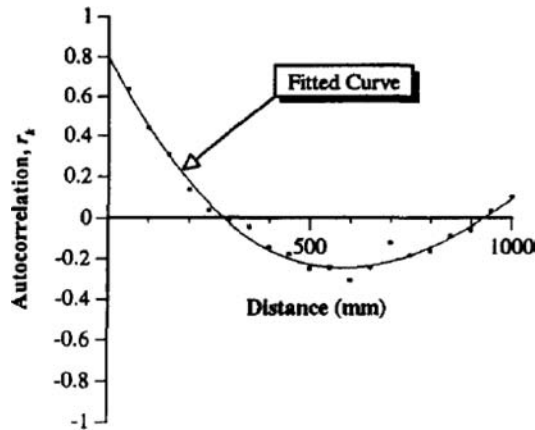


Figure 46. Example application of Baecher's procedure for the estimation of random measurement error (Jaksa et al. 1997).

are not point measurements, but are representative parameters of the extent of the volume in which such failure occurs. The magnitude of the spatial averaging effect is greater than the magnitude of small-scale variation (Vanmarcke 1983). Hence, in the context of practical variability analyses, the attribution of small-scale variations to soil variability or measurement error is of limited relevance.

3.7.1 Baecher's procedure

Baecher (1982) proposed an indirect method for the quantification of small-scale variability. The method consists in the back-interpolation of the sample autocorrelation function (with the exclusion of the origin) to zero separation distance of the autocorrelation plot. A zero-lag autocorrelation coefficient r_0 . Figure 46 (Jaksa et al. 1997) illustrates an application of the method.

3.7.2 Semivariogram nugget

As described in §3.5.6.2, the semivariogram nugget C_0 provides a quantitative measure of the nugget effect, which is related to small-scale variability.

3.7.3 Autocorrelation nugget

Jaksa et al. (1997) defined the autocorrelation function nugget R_0 as the difference between unity and the autocorrelation intercept from Baecher's procedure r_0 :

$$R_0 = 1 - r_0 \quad (31)$$

The autocorrelation function (ACF) nugget provides a measure of the magnitude of small-scale variability, i.e. the residual autocorrelation at separation distances smaller than measurement spacing. It is conceptually analogous to the semivariogram nugget, but is easier to interpret numerically as it ranges from 0 to 1.

Jaksa et al. (1997) investigated the effect of trend removal and sample spacing on the autocorrelation nugget of profiles of cone tip resistance in the highly homogeneous Keswick clay. Table 19 illustrates the dependence of weak stationarity, fit of trend (given by the coefficient of determination R^2) and ACF nugget on the degree of the polynomial trend removed from the source profiles. The ACF nugget increases monotonically with increasing complexity of the polynomial trend function.

Table 20 (Jaksa et al. 1997) illustrates the dependence of the ACF nugget on variations in sample spacing of profiles achieved by sampling source data at differing intervals. While the lower bounds in the range of ACF nuggets do vary, the upper bounds increase significantly with sample spacing.

4 RANDOM FIELD MODELLING OF SPATIAL VARIABILITY

The modelling of spatial correlation using autocorrelation function or semivariogram does not allow full representation of the effects of soil variability on the performance and reliability of spatially random

Table 19. Dependence of weak stationarity, fit of trend and ACF nugget on removed polynomial trend for cone tip resistance data in the vertical direction (adapted from Jaksa et al. 1997).

Degree of polyn. trend	Weak stationarity	Degree of trend fit (R^2)	ACF nugget [%]
none	No	—	5
1	No	0.08	6
2	Yes	0.72	12
3	Yes	0.73	12
4	Yes	0.77	13
5	Yes	0.82	14
6	Yes	0.85	22

Table 20. Dependence of ACF nugget on sample spacing for cone tip resistance data in the vertical direction (adapted from Jaksa et al. 1997).

Sample spacing [mm]	Number of data sets	ACF nugget [%]
5	1	10
10	2	10–11
20	4	5–7
50	5	3–24
100	5	18–60
200	5	3–62

geotechnical systems. Important effects such as the *spatial averaging effect* and the existence of a *critical spatial correlation* require additional operations and parameters.

Spatial variability can be expressed concisely by means of *random field theory*. In most general terms, a *random field* is essentially a random (or *stochastic*) process consisting of an indexed (i.e. ordered according to one or more reference dimension) set of random variables (Vanmarcke 1983).

In an applicative geotechnical perspective, random field theory is important for two main reasons: first, it provides a vehicle for incorporating spatial variation in engineering and reliability models (e.g. Uzielli et al. 2005a); second, through geostatistics, it provides powerful statistical results which can be used to draw inferences from field observations and plan spatial sampling strategies (e.g. Jaksa et al. 2005).

In the present chapter, the spatial averaging effect and variance reduction due to spatial averaging are addressed. The *scale of fluctuation* and the *coefficient of variation of inherent variability* are introduced (among others available in the literature) as concise descriptors of a random field in the second-moment sense. A review of calculation methods for such parameters is provided, as well as selected literature data.

It is anticipated here that variance reduction alone cannot explain the behaviour and reliability of complex geotechnical systems. As will be discussed in Section 7, actual failure mechanisms in 2D or 3D geotechnical problems will involve failure surfaces which do not correspond to the pre-determined ones assumed by traditional approaches (e.g. circular surfaces in some slope stability methods). At present, simulation seems the only viable brute-force solution. Details and examples are discussed in Section 7.

4.1 The issue of scale in random field modelling

It has been stated previously that the finite-scale correlation approach is preferable, at present, for practical evaluation of the spatial variability of soil properties. With reference to finite-scale approaches, Vanmarcke (1983) stated that “*The question of scale is of paramount importance in practical applications of random field theory. A phenomenon that appears deterministic on the micro-scale [...] may at a larger scale exhibit highly variable properties that call for probabilistic description. On an even larger scale, these same structures may be embedded in objects amenable to characterisation in terms of average [...] properties. [...] In other words, there is usually a lower and an upper bound on the range of dimensions within which a random field model has practical value.*”

It is of interest to investigate the physical reasons which support the hypothesis of the existence of upper and lower bounds to spatial correlation in the context of random field modelling. An upper bound to spatial correlation is given by the always finite extension of any sampling domain. As has been discussed in greater detail in §3.7, it is virtually impossible to distinguish small-scale variation from measurement

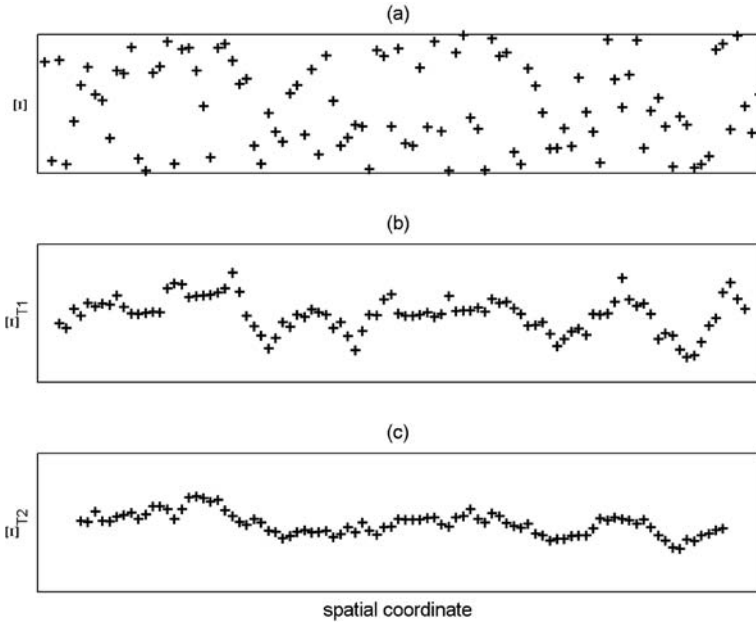


Figure 47. Representation of spatial averaging of random processes measured at discrete, constant spatial interval: (a) non-averaged random process; (b) same random process averaged over an interval T_1 ; (c) same random process averaged over an interval $T_2 > T_1$. Note: x-axis and y-axis scales are the same in all diagrams.

error. Hence, random field models need only provide information about random variables on a scale which is sufficient to represent behaviour under at least some amount of spatial averaging.

4.2 Variance reduction due to spatial averaging

An implicit manifestation of spatial correlation which is commonly encountered in geotechnical engineering practice is that the representative value of any soil property depends on the volume concerned in the problem to be solved. With reference to a given soil unit and to a specific problem, the geotechnical engineer is trained to define the design values of relevant parameters on the basis of the magnitude of the volume of soil governing the design.

Any laboratory or in-situ geotechnical measurement includes some degree of spatial averaging in practice, as tests are never indicative of point properties, but, rather, are used to represent volumes of soil.

Sets of measurements are always *discrete* as there is always some spatial separation between two measurement locations. The local average Ξ_T of a discrete random process Ξ over a spatial interval T is defined by

$$\Xi_T = \frac{1}{T} \sum_{i=1}^n \xi_i \Delta z \quad (32)$$

in which n is the number of measurements in T and Δz is the (constant) discrete measurement interval. Figure 47 provides an example of the effect of spatial averaging: in (a), the non-averaged process (i.e. a generic set of residuals) is shown; in (b) and (c), the same process is shown following spatial averaging for a spatial interval T_1 and T_2 , respectively, with $T_2 > T_1$.

The spatial averaging effect results in a reduction of the effect of spatial variability on the computed performance because the variability is averaged over a volume, and only the averaged contribution to the uncertainty is of interest as it is representative of the “real” physical behaviour (e.g. Lacasse & Nadim 1996).

Figure 48 provides an example of the variance reduction effect due to spatial averaging of a generic parameter in the two-dimensional case. The resulting 3 histograms show that the mean varies little due to spatial averaging (depending on the distribution of the process), but the variance decreases significantly from the “1 × 1 block” (non-averaged) case to the “5 × 5 block” and to the “10 × 10 block” case.

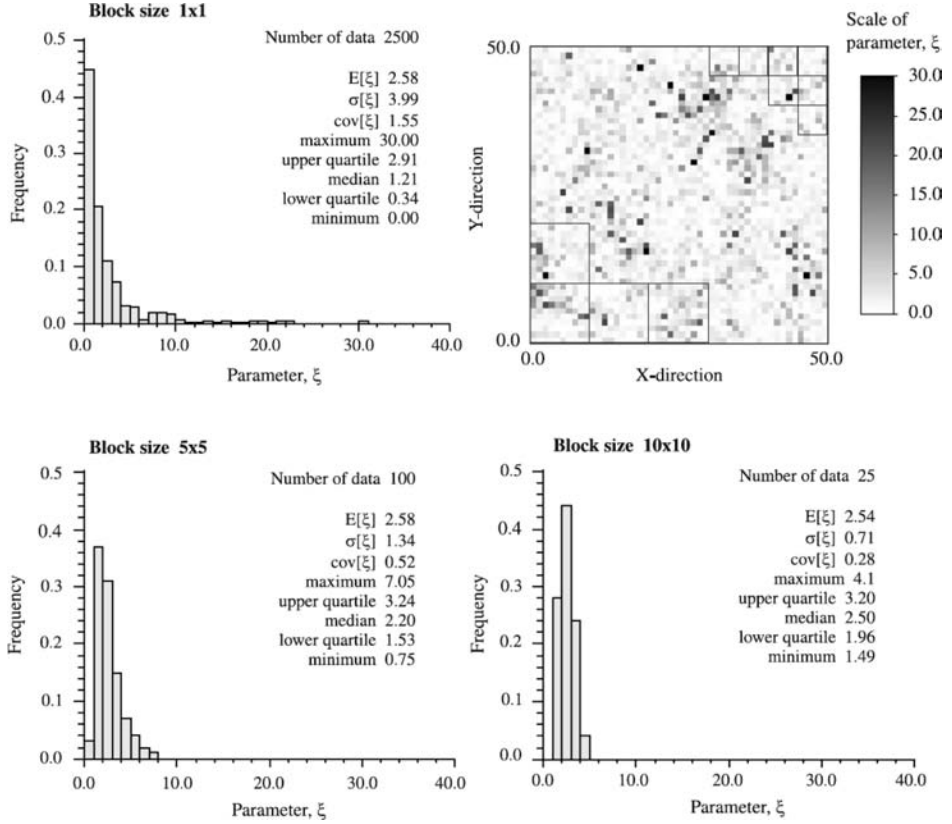


Figure 48. Effect of spatial averaging on variance reduction (El-Ramly et al. 2002).

The reduction in variance due to spatial averaging effect can be represented by the *variance function* (Vanmarcke 1983), which expresses the ratio of the variance of a spatially averaged random process to that of the same (non-averaged) process. In other words, it provides a measure of the reduction of the point variance under spatial averaging as a function of the spatial averaging distance T :

$$\Gamma^2(T) = \frac{s_{\Xi T}^2}{s_{\Xi}^2} \quad (33)$$

in which $s_{\Xi T}$ is the variance of the trend function and s_{Ξ} is the total variance of the data set. The variance reduction functions of theoretical autocorrelation models can also be expressed concisely by mathematical expressions (e.g. Vanmarcke 1983). However, as for the autocorrelation function and semivariogram, the limited size of data sets makes it necessary to estimate the variance reduction based on samples which are always limited in size and are composed of measurements obtained at discrete spatial intervals. Given a series of n equally spaced observations over a sampling domain of size T , the discrete approximation of the i -th value of the sample variance reduction function is

$$\hat{\Gamma}^2_i = \frac{1}{s_{\Xi}^2(n-i+1)} \sum_{j=1}^{n-i+1} (\xi_{i,j} - m_{\Xi})^2 \quad (34)$$

for $i = 1, 2, \dots, n$, where

$$\xi_{i,j} = \frac{1}{i} \sum_{k=j}^{j+i-1} \xi_k \quad (35)$$

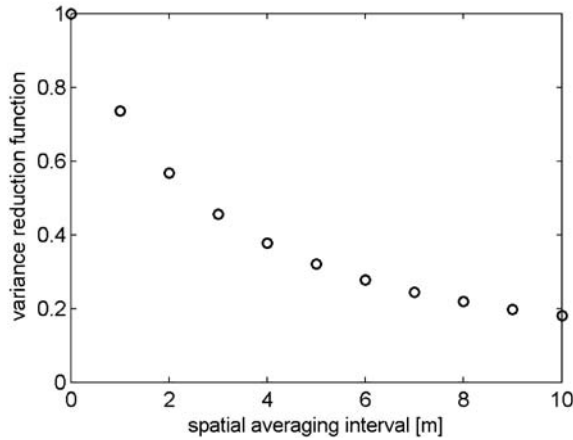


Figure 49. Example of a discrete variance reduction function.

for $j = 1, 2, \dots, n - i + 1$. Figure 49 shows a typical variance reduction function calculated in the discrete case using Eq. (34) and (35). From the figure it is inferred, for instance, that the variance of the random process averaged over 6 m is equal to the variance of the non-averaged process multiplied by 0.28.

The condition of at least weak stationarity is necessary because the variance reduction function expresses the reduction in the variance of an averaged process as a function of the averaging interval only, and not on the spatial position of a subset of the random process itself (i.e. whether the spatial averaging occurs in the initial or terminal section of a profile).

The results obtained for the one-dimensional case may be extended to the two- and three-dimensional cases by assuming a separable, independent correlation structure for each spatial dimension. Hence, the total variance reduction can be calculated as the product of the variance reductions in the individual dimensions. The reader is referred to Vanmarcke (1983) and Elkateb et al. (2003a) for further insights into multi-dimensional variance reduction.

4.3 Scale of fluctuation

Vanmarcke (1983) defined the *scale of fluctuation* as the proportionality constant in the limiting expression of the variance function, i.e. as the value assumed by the product of the variance function and the spatial averaging extension when the latter tends to infinity (or, in practice, becomes large):

$$\delta = \lim_{T \rightarrow \infty} \Gamma^2(T) \cdot T \quad (36)$$

In finite-scale models, the scale of fluctuation, δ , is a concise indicator of the spatial extension of the correlation structure. Within separation distances smaller than the scale of fluctuation, the deviations from the trend function are expected to show relatively strong correlation. When the separation distance between two sample points exceeds the scale of fluctuation, it can be assumed that little correlation exists between the fluctuations in the measurements.

The scale of fluctuation is an extremely important parameter for the representation of a random field. Actually, a random field is described (in the second-moment sense) by its mean, standard deviation and scale of fluctuation, as well as by a functional form for the autocorrelation function. The scale of fluctuation is also useful to approximate the variance reduction function, thereby allowing to avoid use of Eq. (34) and (35) (see §4.6).

The scale of fluctuation of a random process does not always exist. Vanmarcke (1983) showed that the condition for the existence of the scale of fluctuation is (in simple, qualitative terms) is that the autocorrelation function converges to zero with increasing separation distance.

Other concise descriptors of spatial correlation have been employed in the geotechnical literature. These include *autocorrelation distance* and *effective semivariogram range*. The interested reader is referred to Jaksa (1995), Lacasse & Nadim (1996) and Elkateb et al. (2003a). As each represents distinct characteristics of spatial correlation structure, the respective numerical values are different for a given

Table 21. Scale of fluctuation for selected autocorrelation models.

Model	Scale of fluctuation
Single exponential (SNX)	$\delta = 2/k_{SNX}$
Cosine exponential (CSX)	$\delta = 1/k_{CSX}$
Second-order Markov (SMK)	$\delta = 4/k_{SMK}$
Squared exponential (SQX)	$\delta = \sqrt{\pi}/k_{SQX}$

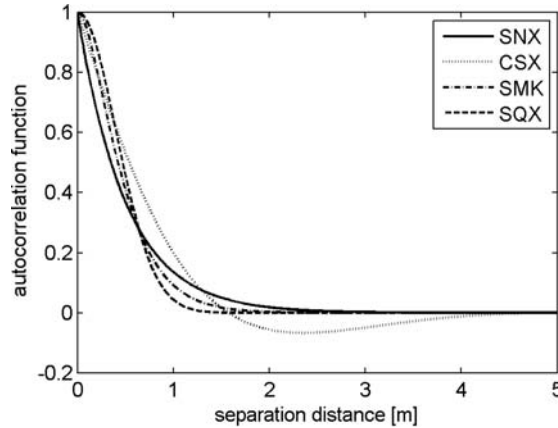


Figure 50. Autocorrelation models for scale of fluctuation equal to 1.00 m.

autocorrelation function (or semivariogram); hence, care should be taken as to avoid inadvertently using one in place of the other.

A variety of techniques for the estimation of the scale of fluctuation are available in the geotechnical literature. A number of these are presented briefly in the following.

4.3.1 Fitting of autocorrelation models (AMF)

The scale of fluctuation can be calculated from the fitting of theoretical autocorrelation models to empirically calculated sample autocorrelation functions. When the best-fit autocorrelation model has identified by regression, the characteristic model parameter can be obtained as explained in §3.5.6.1. The scale of fluctuation may then be evaluated by applying the appropriate analytical relationship with the characteristic model parameter of the best-fit model among those shown in Table 21.

Figure 50 provides a comparative plot of the SNX, CSX, SMK and SQX autocorrelation models for a unit scale of fluctuation, i.e. for $k_{SNX} = 2$; $k_{CSX} = 1$; $k_{SMK} = 4$; and $k_{SQX} = \sqrt{\pi}$. While the models appear to be quite similar, the importance of the different behaviours at low separation distances (i.e. where the autocorrelation coefficients are estimated most reliably) should not be downplayed. With reference to Figure 50, for instance, at a separation distance $\tau = 0.25$ m, the single exponential and square exponential models would provide, respectively, autocorrelation coefficients of 0.61 and 0.82, which indicate considerably different magnitude of spatial correlation! Hence, the identification of the most suitable correlation structure should be performed with utmost care and as rigorously as possible.

Figure 51a (Phoon 2006b) shows a detrended normalized cone tip resistance (q_{cIN}) profile spanning between 14.125 m and 27.975 m in depth. The sampling interval is 2.5 cm. The empirical autocorrelation function can be fitted to a single exponential function or a square exponential function with a scale of fluctuation = 0.98 m (Figure 51b).

The simulated profiles are shown in Figure 52. The low frequency fluctuations are quite accurate for both models. The high frequency fluctuations are over-represented for the single exponential model and under-represented for the square exponential model. The reason is that the rate of decay is too fast for the single exponential model and too slow for the square exponential model.

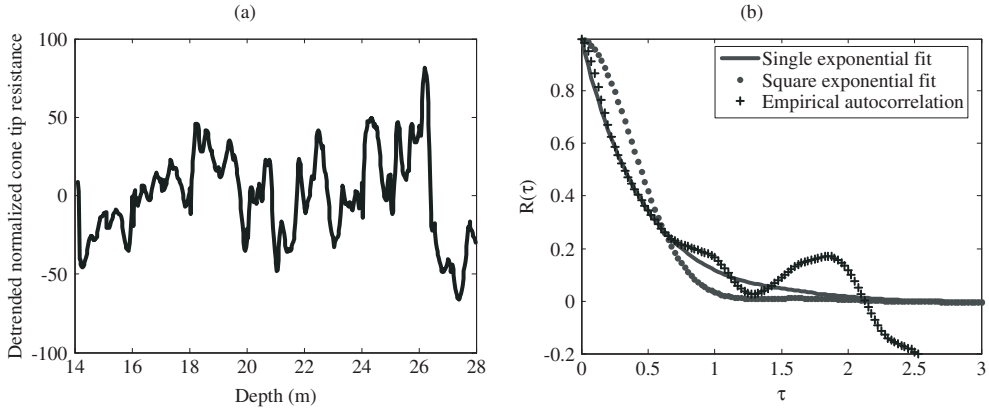


Figure 51. (a) Detrended normalized cone tip resistance profile and (b) Empirical autocorrelation function fitted with two theoretical models (Phoon 2006b).

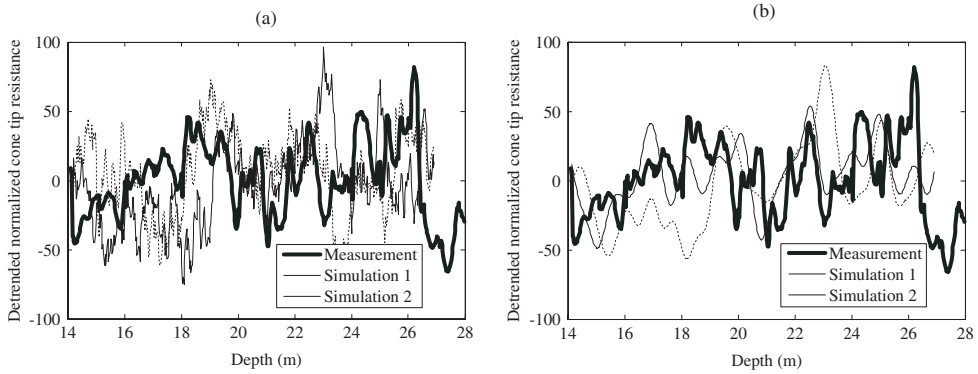


Figure 52. Simulated profiles from (a) Single exponential model and (b) Square exponential model (Phoon 2006b).

4.3.2 Integration of sample autocorrelation function (SAI)

Vanmarcke (1983) showed that the scale of fluctuation of a random process, if it exists, is also given by the area under the autocorrelation function or, since the latter is symmetric about the origin, to twice the area below the positive-separation distance semi-function.

In the discrete case, this can be approximated by Eq. (37):

$$\delta = 2 \sum_{i=2}^m \frac{(\hat{R}_{i-1} + \hat{R}_i)}{2} \cdot \Delta\tau = \Delta\tau \sum_{i=2}^m (\hat{R}_{i-1} + \hat{R}_i) \quad (37)$$

where $\hat{R}$ is the estimated sample autocorrelation function (defined in §3.5.6.1) and $\Delta\tau$ is the constant separation distance between two consecutive measurements. As the scale of fluctuation is not well defined for autocorrelation functions which do not converge to zero relatively rapidly, m is usually taken to be the index of the separation distance where the sample correlation function first becomes negative (Jaksa 1995). An example application of the SAI technique to a detrended q_{c1N} profile is shown in Figure 53.

4.3.3 Bartlett limits method (BLM)

Jaksa (1995) observed that the scale of fluctuation of cone tip resistance in clays was very well approximated by *Bartlett's distance* r_B , i.e. the separation distance corresponding to the first intersection of the sample autocorrelation function with Bartlett's limits [see Eq. (29)]. An advantage of this method is that the estimated scale of fluctuation is not affected by measurement interval (Jaksa 1995).

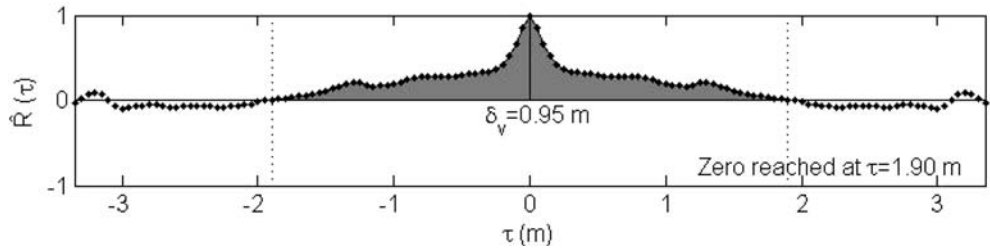


Figure 53. Application of the SAI technique for the estimation of the scale of fluctuation (Uzielli 2004).

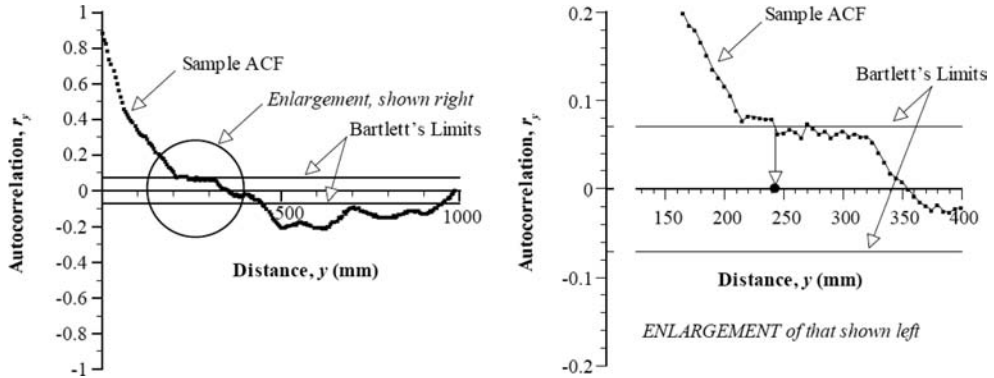


Figure 54. Example of application of the BLM technique for the estimation of the scale of fluctuation (Jaksa 2006).

Uzielli (2004) asserted that only scales of fluctuation obtained from sample autocorrelation functions definitively oscillating inside Bartlett's limits should be accepted. An application of the BLM technique to cone penetration resistance data (Jaksa 2006) is shown in Figure 54.

4.3.4 Simplified method (VXP)

Vanmarcke (1977) proposed the following approximate relationship for evaluating the scale of fluctuation:

$$\delta \approx \sqrt{\frac{2}{\pi} \bar{\Delta}} \quad (38)$$

where

$$\bar{\Delta} = \frac{1}{n_c} \sum_{i=1}^{n_c} \Delta_i \quad (39)$$

is the average distance between the intersections of the fluctuating component and the trend of a given profile (see Figure 55).

4.3.5 Fluctuation function method (VRF)

Based on the definition by Vanmarcke (1983), the scale of fluctuation can be estimated in practice as the limit value of the *fluctuation function*, i.e. the product of the variance function and the separation distance when the latter becomes “large”. Such procedure has been employed in several studies (e.g. Wickremesinghe & Campanella 1993; Cafaro & Cherubini 2002; Uzielli 2004).

A unique criterion to identify “large separation distance” is not available. Wickremesinghe & Campanella (1993) suggested that the separation distance corresponding to the point of inflection of the variance reduction function may be used; Uzielli (2004) adopted the separation distance corresponding to a variance reduction coefficient of 0.15.

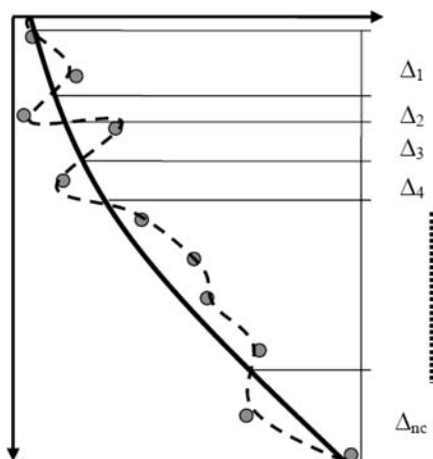


Figure 55. Mean-crossings approximation for the estimation of the scale of fluctuation (Vanmarcke 1977).

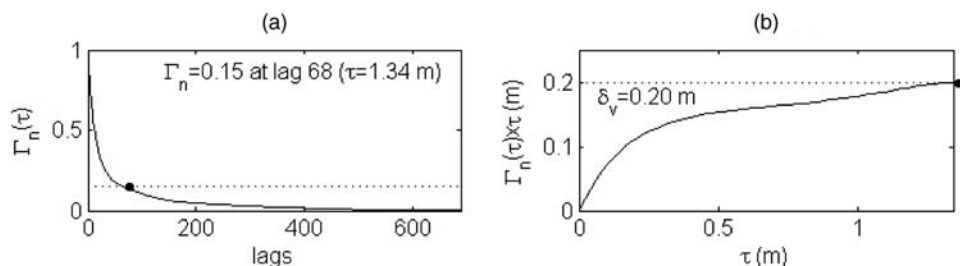


Figure 56. Application of the VRF technique for the estimation of the scale of fluctuation: (a) variance reduction function; (b) fluctuation function (Uzielli 2004).

Fenton (1999a) showed that the sample variance function becomes unbiased only if the averaging region grows large and the correlation function decreases rapidly within the averaging region itself. Like the sample correlation function, the sample variance function is really only a useful estimate of second-moment behaviour for short-scale processes, i.e. processes whose spatial correlation distance is small in comparison with their spatial extension.

An example application of the VRF technique to a detrended cone penetration resistance profile (Uzielli 2004) is shown in Figure 56. In the example, the variance reduction function attains a value of 0.15 at 68 lags (i.e. data points) from the origin, equivalent to a separation distance of 1.34 m because the measurement interval is 2 cm. Hence, the scale of fluctuation is taken as the value of the fluctuation function at $\tau = 1.34$ m.

4.3.6 Estimation from semivariogram range (SVR)

Similarly to the AMF method for the autocorrelation functions, the scale of fluctuation can be calculated from the fitting of theoretical semivariogram models to empirically calculated sample semivariograms. When the best-fit semivariogram model has been identified by regression, the characteristic model parameter can be obtained, and the scale of fluctuation may then be evaluated by applying the appropriate analytical relationship with the characteristic model parameter of the best-fit model among those shown in Table 22.

4.4 Factors influencing the scale of fluctuation

The scale of fluctuation is not an inherent property of a soil parameter. Estimated values of the scale of fluctuation are closely bound to the estimation methodology, as they depend at least on: (a) the spatial

Table 22. Relation between scale of fluctuation and characteristic semivariogram model parameters (Elkateb et al. 2003a).

Model	Scale of fluctuation
SPH	$\delta = 0.55a_{SPH}$
EXP	$\delta = 2a_{EXP}/3$
GAU	$\delta = a_{GAU}$

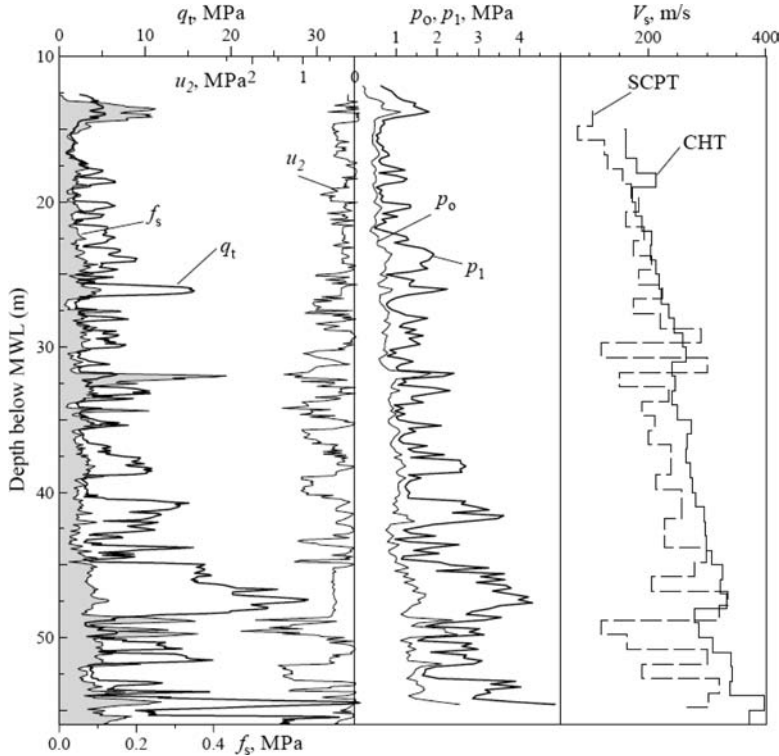


Figure 57. Profiles from cone penetration, flat dilatometer, seismic cone and cross-hole testing from the Malamocco site in the Venice lagoon, showing the effect of the difference in measurement interval (Simonini et al. 2006).

direction [e.g. horizontal, vertical]; (b) the measurement interval in the source data; (c) the type of trend which is removed during decomposition; (d) the method of estimation of the scale of fluctuation from residuals [e.g. fluctuation function method vs. autocorrelation model fitting]; and (e) modelling options from the specific estimation method [e.g. choice of best-fit autocorrelation model].

Point (a) refers to the inherent anisotropy of soils and in-situ soil masses. From a geological and geomorphological perspective, it is intuitive that soil formation and modification processes, as well as factors contributing to the definition of the in-situ state (e.g. stress) would result in a greater heterogeneity of soil properties in the vertical direction and, hence, in a weaker spatial correlation. As will be shown in §4.5, the scale of fluctuation of a given soil property in the vertical direction is generally much smaller than in the horizontal direction.

Regarding point (b), research has shown that the scale of fluctuation depends on the measurement interval. More specifically, it tends to increase with sample spacing. In geotechnical site characterisation, measurement spacing varies significantly among testing methods. Figure 57 (Simonini et al. 2006) illustrates the difference in resolution of cone penetration tests, flat dilatometer tests, seismic cone and cross-hole tests from the Malamocco site in the Venice lagoon.

Cafaro & Cherubini (2002) evaluated the scale of fluctuation of data sets of cone tip resistance in two clay layers from a Southern Italian site using the VRF method. The sets were sampled at different

Table 23. Vertical scales of fluctuation of cone tip resistance calculated using quadratic detrending and the VRF method for different measurement intervals and for soundings G1-G15 in two clay layers in Southern Italy (Cafaro & Cherubini 2002).

Spacing [m]	G1 [m]	G3 [m]	G6 [m]	G7 [m]	G15 [m]	Mean [m]
Upper clay						
0.20	0.20	0.40	0.21	0.40	0.44	0.33
0.40	0.20	0.34	0.26	0.44	0.51	0.35
0.60	0.19	0.35	0.24	0.52	0.47	0.36
Lower clay						
0.20	0.54	0.29	0.72	0.27	0.19	0.40
0.40	0.64	0.39	0.67	0.28	0.23	0.44
0.60	0.41	0.37	0.96	0.28	0.53	0.51

Table 24. Influence of trend removal on the scale of fluctuation of cone tip resistance of an Italian clay calculated using the VRF method (Cafaro & Cherubini 2002).

Sounding	Scale of fluctuation [m] for linear trend removal	Scale of fluctuation [m] for quadratic trend removal
G1	0.42	0.20
G3	0.40	0.40
G6	0.60	0.21
G7	0.96	0.40
G15	0.47	0.44

measurement intervals: 0.2, 0.4 and 0.6 m. The results, reported in Table 23, indicate that the scale of fluctuation tends to increase with measurement interval.

Trend removal generally results in a decrease in the estimated scale of fluctuation, because, as seen previously, the trend accounts for some spatial correlation. Moreover, for a given estimation methodology, the scale of fluctuation decreases with increasing complexity of the trend. Table 24, which summarises the results of an investigation by Cafaro & Cherubini (2002) on cone resistance values measured at 0.20m intervals, provides an example of such phenomenon.

Regarding the influence of the estimation method, Uzielli (2004) compared the estimates of the scale of fluctuation of profiles of normalised cone tip resistance using the SAI, VXP, BLM, VRF and AMF estimation methods described in §4.3. The results are plotted in Figure 58. Each stack of values refers to a weakly stationary, physically homogeneous soil unit. Measurement intervals for the different profiles ranged from 0.002 to 0.10 m. By examination of Figure 58, it may be stated that none of the models rank consistently above or below the others; however, in some cases the scatter among estimates from different methods is significant.

Jaksa (1995) investigated the relationship between Bartlett's distance and the scale of fluctuation (obtained by autocorrelation model fitting) for Keswick clay. The strong regression fit, shown in Figure 59, attests for the good agreement between the methods in the specific case.

Based on the above observations, it may be stated that numerical values of the scale of fluctuation should thus be viewed with extreme caution. Strictly speaking, they should only be used for inferential purposes if the measurement interval, the removed trend type and the estimation methodology are explicitly reported along with the numerical estimate.

Despite the marked dependence on the estimation procedure, research has shown (e.g. Phoon & Kulhawy 1999; Uzielli et al. 2005a, 2005b) that, if the vertical scale of fluctuation of different soil types (e.g. cohesive vs. cohesionless) and the same conditions are maintained with regards to (a)–(e), a dependency on soil type is generally observed.

Figure 60 reports some of the results of an investigation by Uzielli et al. (2005a) on random field characterisation of stress-normalised cone penetration parameters, namely the normalised, dimensionless cone penetration resistance q_{c1N} and the normalised friction ratio F_R .

Scales of fluctuation in the vertical direction were estimated by fitting autocorrelation models to linearly detrended data from highly physically homogeneous soil units from five regional sites worldwide. These were identified using a procedure described in Section 5. In the plots, each point represents one soil unit. As normalised cone tip resistance is related to soil type (with resistance decreasing with increasing cohesive behaviour), the plots can be useful in investigating the existence of a soil-type effect on vertical

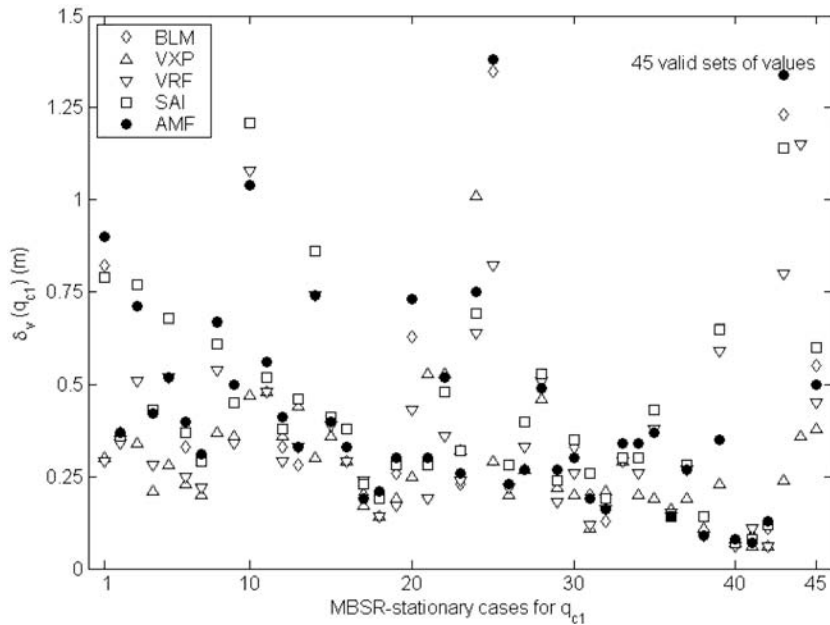


Figure 58. Estimates of the vertical scale of fluctuation for q_{c1} using BLM, VXP, VRF, SAI and AMF (Uzielli 2004).

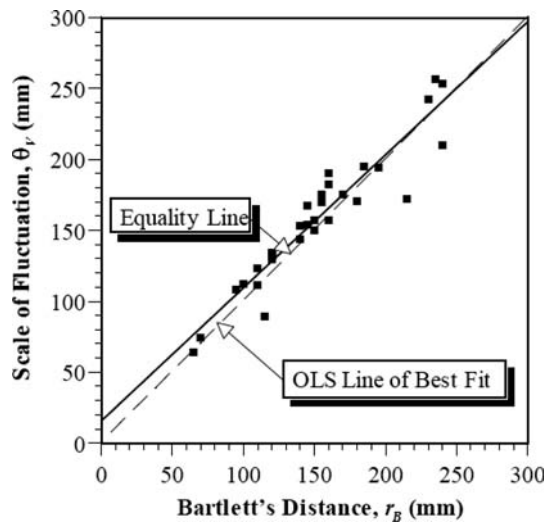


Figure 59. Empirical relationship between the scale of fluctuation and Bartlett's distance for Keswick clay (Jaksa 1995).

spatial correlation. There appears to be a positive correlation between mean value and vertical scale of fluctuation, indicating that the vertical spatial correlation structure of cohesionless soils would be generally higher than that of cohesive soils.

Figure 60a plots the vertical scale of fluctuation vs. the mean value of normalised cone tip resistance, with points categorised by regional site. With the exception of the Treasure Island data (TSI), there seems to be no site-specificity in the results. Figure 60b shows the same data, but points are categorised by best-fit autocorrelation model: it appears that best-fit autocorrelation models are not associated to soil type, and that none of the models provides consistently higher or lower vertical scales of fluctuation.

Similar results were obtained by Uzielli et al. (2005b) while investigating random field parameters of Robertson's soil behaviour classification index, I_c (e.g. Robertson & Wride 1998), which maps the usual

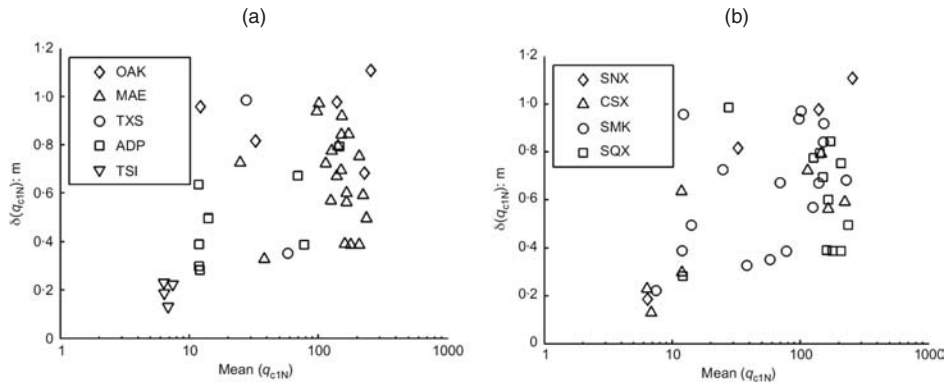


Figure 60. Vertical scale of fluctuation of stress-normalised cone tip resistance vs. mean value of resistance in physically homogeneous soil unit: (a) by site; and (b) by best-fit autocorrelation model (Uzielli et al. 2005a).

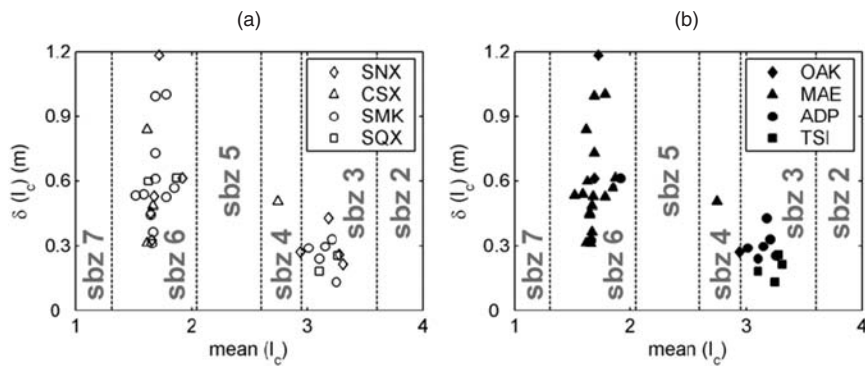


Figure 61. Vertical scale of fluctuation of Robertson's CPT soil behaviour classification index I_c vs. mean value of I_c : (a) by best-fit autocorrelation model; and (b) by site (Uzielli et al. 2005b).

two-dimensional CPT-based soil classification zones onto a one-dimensional scale and is calculated from q_{c1N} and F_R . Such parameter is a concise descriptor of soil type in terms of mechanical behaviour. Ranges of I_c correspond to different soil behaviour zones (SBZs), with higher values (pertaining to SBZs 2 and 3) corresponding to cohesive-behaviour soils, intermediate values (SBZs 4 and 5) to intermediate-behaviour soils and lower values (SBZs 6 and 7) to cohesionless-behaviour soils. Again, the generally stronger vertical spatial correlation structure of cohesionless soils may be seen in Figure 61.

The dependency of spatial correlation on soil type could have interesting geotechnical implications. In the case of cone penetration testing, for instance, past research (e.g. Teh & Houlsby 1991) indicates that the extent of the failure zone increases with increasing soil stiffness; thus, it may be postulated that the influence zone affecting cone tip resistance is larger in sand (and a greater number of adjacent measurements would be strongly correlated), while sleeve friction is only affected by the adjacent soil regardless of soil type (with consequently weaker spatial correlation). Hence, the results obtained by Uzielli et al. (2005a, 2005b) could be explained in terms of the physical phenomena involved in penetration. Other geotechnical testing methods could provide similar results and allow for increasingly confident inferences; unfortunately, the literature is surprisingly limited at present.

4.5 Scale of fluctuation: literature review

A synthetic tabular review of literature values of the horizontal and vertical scale of fluctuation of a number of geotechnical parameters is provided in Table 25. The table relies significantly on a number of data collection efforts by Phoon et al. (1995), Jaksa (1995), Lacasse & Nadim (1996), Phoon & Kulhawy (1999a) and Jones et al. (2002). More complete descriptions of the source data (including, for instance, measurement interval) are provided, when available, in such references. More recent efforts addressing the estimation of random field parameters include Elkateb et al. (2003b) and Uzielli et al. (2005a).

Table 25. Literature values for the horizontal (δ_h) and vertical (δ_v) scale of fluctuation of geotechnical parameters.

Property*	Soil type	Testing method**	δ_h (m)	δ_v (m)
s_u	Clay	Lab. testing	–	0.8–8.6
s_u	Clay	VST	46.0–60.0	2.0–6.2
q_c	Sand, clay	CPT	3.0–80.0	0.1–3.0
q_c	Offshore soils	CPT	14–38	0.3–0.4
$1/q_c$	Alluvial deposits	CPT	–	0.1–2.6
q_t	Clay	CPTU	23.0–66.0	0.2–0.5
q_{c1N}	Cohesive-behaviour soils	CPT	–	0.1–0.6
q_{c1N}	Intermediate-behaviour soils	CPT	–	0.3–1.0
q_{c1N}	Cohesionless-behaviour soils	CPT	–	0.4–1.1
f_s	Sand	CPT	–	1.3
f_s	Deltaic soils	CPT	–	0.3–0.4
F_R	Cohesive-behaviour soils	CPT	–	0.1–0.5
F_R	Intermediate-behaviour soils	CPT	–	0.1–0.6
F_R	Cohesionless-behaviour soils	CPT	–	0.2–0.6
I_c	Cohesive-behaviour soils	CPT	–	0.2–0.5
I_c	Intermediate-behaviour soils	CPT	–	0.6
I_c	Cohesionless-behaviour soils	CPT	–	0.3–1.2
N	Sand	SPT	–	2.4
w	Clay, loam	Lab. testing	170.0	1.6–12.7
w_L	Clay, loam	Lab. testing	–	1.6–8.7
γ'	Clay	Lab. testing	–	1.6
γ	Clay, loam	Lab. testing	–	2.4–7.9
e	Organic silty clay	Lab. testing	–	3.0
σ'_p	Organic silty clay	Lab. testing	180.0	0.6
K_S	Dry sand fill	PLT	0.3	–
$\ln(D_R)$	Sand	SPT	67.0	3.7
n	Sand	–	3.3	6.5

* s_u = undrained shear strength; q_c = cone tip resistance; q_t = corrected cone tip resistance; q_{c1N} = dimensionless, stress-normalised cone tip resistance; f_s = sleeve friction; F_R = stress-normalised friction ratio; I_c = CPT soil behaviour classification index; N = SPT blow count; w = water content; w_L = liquid limit; γ' = submerged unit weight; γ = unit weight; e = void ratio; σ'_p = preconsolidation pressure; K_S = subgrade modulus; D_R = relative density; n = porosity.

** VST = vane shear testing; CPT = cone penetration testing; CPTU = piezocone testing; SPT = standard penetration testing; PLT = plate load testing.

It is possible to appreciate the strong anisotropy in spatial correlation strength, horizontal scales of fluctuation being much larger than corresponding vertical scales. Given the strong set-specificity of the scale of fluctuation and its dependence on test type and estimation method, data should not be exported uncritically to other sites.

4.6 Approximation of the variance reduction function

For a random process whose scale of fluctuation δ is known, the sample variance reduction function over a spatial averaging interval T can be approximated, for practical applications, by (Vanmarcke 1983):

$$\Gamma^2(T) = \begin{cases} 1 & \text{for } T \leq \frac{\delta}{2} \\ \frac{\delta}{T} \left(1 - \frac{\delta}{4T} \right) & \text{for } T \geq \frac{\delta}{2} \end{cases} \quad (40)$$

4.7 Coefficient of variation of inherent variability

For data sets satisfying the condition of weak stationarity, the dimensionless *coefficient of variation of inherent variability* (Phoon & Kulhawy 1999a), η_{ω} , is obtained by normalising the standard deviation of

the residuals s_ω with respect to the value of the trend function at the midpoint of the homogeneous soil unit under investigation, m_t :

$$\eta_\omega = \frac{s_\omega}{m_t} \quad (41)$$

where

$$s_\omega = \sqrt{\frac{1}{n-1} \sum_{i=1}^n \omega_i^2} \quad (42)$$

is the standard deviation of the inherent soil variability, n is the number of data points in the profile, and ω_i is the i -th term in the set of residuals. Implicitly, Eq. (42) assumes (as generally occurs) that residuals are a zero-mean set.

The coefficient of variation of inherent variability is a measure of the variance of residuals, and implicitly depends on the spatial correlation structure because it is associated with a specific trend and the resulting set of residuals. Different values of η_ω result from the same set if different trends are removed.

It should be noted that the mean value of the trend is required for the calculation of η_ω . This implies that an estimated value of η_ω is meaningful over an entire spatial extension if m_t is representative of the trend in the spatial extension itself. In other words, it is really only meaningful to estimate the coefficient of variation of inherent variability if the range in trend values in the homogeneous soil unit of interest is not too large.

While the scale of fluctuation investigates the magnitude of spatial correlation in the set of residuals, the coefficient of variation of inherent variability is conceptually related to the magnitude of the dispersion of the residuals about the trend.

4.8 Coefficient of variation of inherent variability: literature review

The coefficients of variation reported in Table 7 (§2.14.2) are generally not calculated using the most rigorous procedure available (i.e. following proper identification of homogeneous soil units, decomposition, identification of correlation structure and assessment of stationarity and application of Equations (41) and (42)). To the writers' best knowledge, the only results in the geotechnical literature which make use of such procedure are those reported by Uzielli (2004) and Uzielli et al. (2005a, 2005b).

Uzielli et al. (2005a) investigated the magnitude of coefficients of variation of inherent variability of stress-normalised cone penetration parameters tip resistance in physically homogeneous soil units. As shown in Figure 62, which reports the results for normalised cone tip resistance, cohesionless-behaviour soil profiles are generally characterised by higher coefficients of variation. This is related to the more erratic nature of CPT profiles in cohesionless soils, and is consistent with the physical phenomena occurring during cone penetration.

Uzielli et al. (2005b) estimated the coefficient of variation of inherent variability for the CPT soil behaviour classification index I_c using data from sites worldwide. Results are reported graphically in Figure 63. The reader is referred to §4.5 for symbols and notation.

As for the scales of fluctuation reported in Figures 60 and 61, it is possible to identify a soil-type effect in Figures 62 and 63, with cohesive-behaviour soils displaying lower values of the coefficient of variation of inherent variability than intermediate- and cohesionless-behaviour soils.

As random field parameters are bound at least to some degree to the characteristics of data (and, hence, to the testing method), other testing methods may provide results which may be qualitatively and quantitatively different. Further studies are warranted to allow more confident statements.

Though comparison with other literature data is not possible at present due to the absence of data concerning stress-normalised cone penetration testing parameters and the soil behaviour classification index, it is to be expected that coefficients of variation obtained using the rigorous procedure should be lower in magnitude than the ones calculated as second-moment statistics of a source data set. The latter can, in fact, be expected to estimate the total variance of the data, thus also the variance in possible spatial trends.

4.9 Probabilistic slope stability analysis accounting for spatial variability: literature examples

The literature examples provided in §2.17 focused on reliability-based analysis using second-moment statistics, distributions of soil properties and correlation among these, but did not account for the spatial

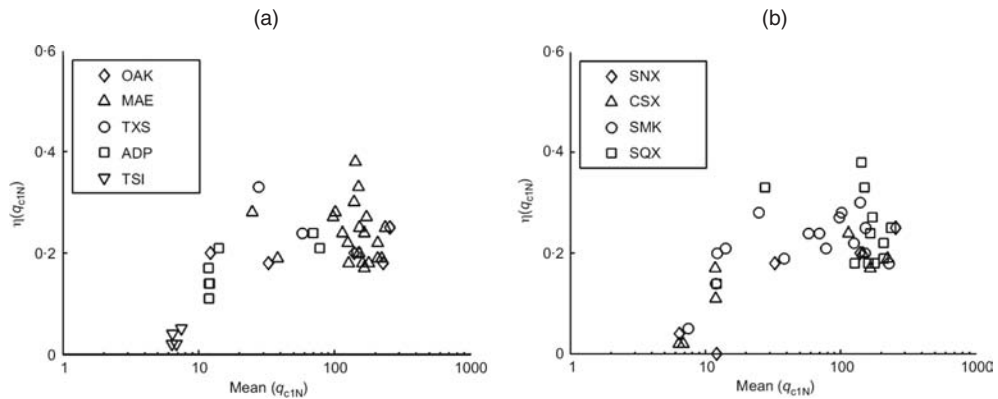


Figure 62. Coefficient of variation of inherent variability of dimensionless stress-normalised cone tip resistance q_{c1N} vs. mean value in physically homogeneous soil unit: (a) by site; and (b) by best-fit autocorrelation model (Uzielli et al. 2005a).

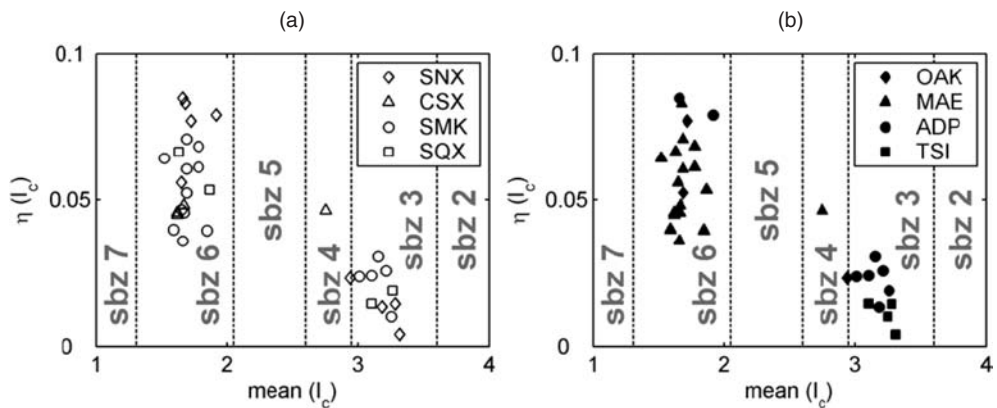


Figure 63. Coefficient of variation of inherent variability of the CPT soil behaviour classification index I_c vs. mean value in physically homogeneous soil unit: (a) by best-fit autocorrelation model; and (b) by site (Uzielli et al. 2005b).

variability. When using Monte Carlo simulation, for instances, each realisation assumed that the sampled values of soil parameters were uniform throughout the slopes. While the probabilistic analyses in §2.17 are undoubtedly a significant improvement over deterministic methods, they may overlook important effects due to spatial averaging. Such effects can only be quantified if random field parameters are defined as well as second-moment statistics and distributions of relevant parameters. A number of studies have addressed the inclusion of spatial variability in slope stability analysis; in the following, selected contributions are reported briefly.

El-Ramly et al. (2003) implemented a probabilistic slope stability approach by El-Ramly et al. (2002). Two candidate slip surfaces (shown in Figure 64) were considered: the deterministic critical slip surface estimated in a preliminary conventional slope analysis; and the minimum reliability index surface obtained from a search algorithm proposed by Hassan & Wolff (1999). The Bishop method of slices was used in the model with the limit equilibrium equations re-arranged to account for the non-circular portion of the slip surface. The spatial variability of the input variables (residual friction angle of the clay-shale; peak friction angle of the sandy till; residual pore pressure ratio in the clay-shale; pore pressure ratios in the sandy till at the middle and at the toe of the slope) along the slip surface are modelled as one-dimensional random fields with an exponentially decaying correlation structure (i.e. SNX model) and a set of common horizontal autocorrelation distances (i.e. *not* the scale of fluctuation) selected from literature values. A lognormal distribution was selected for residual friction angle of the clay-shale; experimentally derived empirical distributions were used for the other parameters. The need to account for variance reduction was bypassed by assigning widths smaller than the autocorrelation distance to

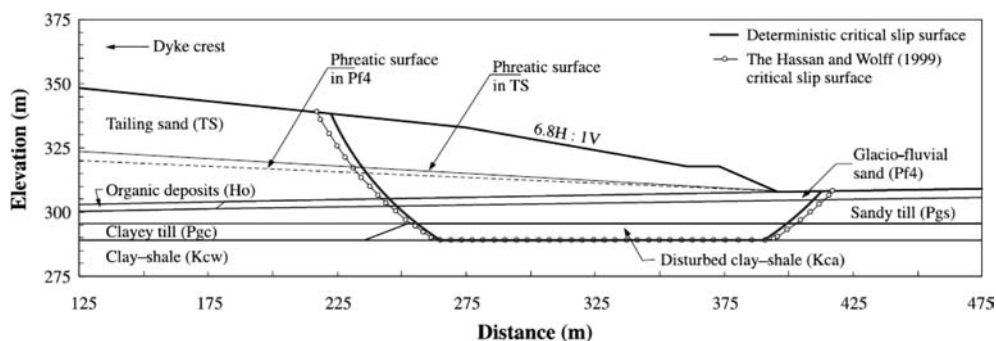


Figure 64. Profile for the probabilistic slope stability analysis of the Syncrude Tailings Dyke by El-Ramly et al. (2003).

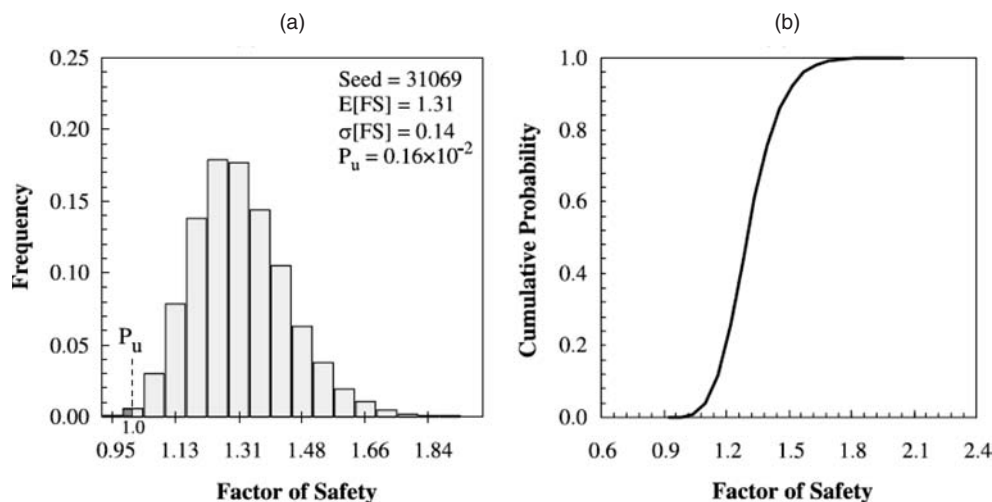


Figure 65. Results of 34000 Monte Carlo simulation realisations for probabilistic slope stability analysis by (El-Ramly et al. 2003): (a) frequency histogram; and (b) cumulative probability curve of factor of safety, considering a horizontal autocorrelation distance of 33 m.

slices. Monte Carlo simulation was used to obtain a probability distribution of the factor of safety for reliability-based analysis.

Figure 65 shows the results of 34000 Monte Carlo simulation realisations assuming a horizontal autocorrelation distance of 33 m, in terms of: (a) frequency histogram of the factor of safety; and (b) cumulative probability curve of the factor of safety. Results are based on the Hassan & Wolff surface, which provided comparatively lower factors of safety than the deterministic slip surface.

El-Ramly et al. (2003) also investigated the sensitivity of the estimated probability of the unsatisfactory performance (i.e. the cumulative probability value corresponding to unit factor of safety) to the value of the autocorrelation distance. It may be observed in Figure 66 that the probability of the unsatisfactory performance increases four-fold as autocorrelation distance increases from 28 m to 38 m.

To investigate the effects of including spatial correlation, El-Ramly et al. (2003) performed simulations without accounting for spatial variability in model parameters. The resulting probability of unsatisfactory performance was one order of magnitude higher than the analysis accounting for spatial variability; hence, in the specific case, spatial variability provides less conservative results.

Griffiths & Fenton (2004) investigated the effects of addressing the effects of spatial averaging in a simulation-based probabilistic analysis. The method of analysis is essentially that referred to in §2.17; however, the reference lognormal probability distribution from which values of normalised strength are sampled is modified accounting for spatial averaging. The main effect on the distribution is perhaps the reduction of both the log-mean and the log-variance of the distribution.

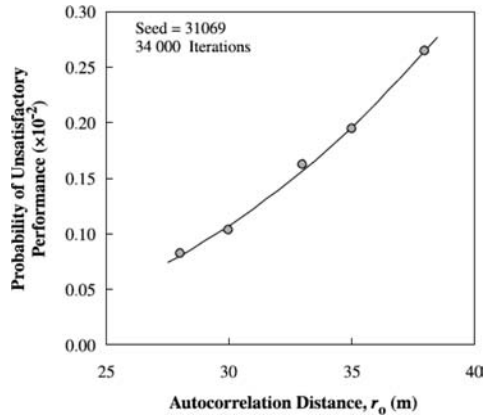


Figure 66. Variation of the probability of unsatisfactory performance with autocorrelation distance (El-Ramly et al. 2003).

5 IDENTIFICATION OF PHYSICALLY HOMOGENEOUS SOIL UNITS

The physical homogeneity of soil units (HSU) is a fundamental prerequisite for variability analyses. Performing variability analyses on soils which are not homogeneous in terms of the property of interest can result in incorrect estimates.

The first issue which is relevant for physical homogeneity assessment is perhaps whether such assessment should occur on a subjective or an objective basis. A purely subjective assessment would rely uniquely on expert judgment. It has been observed (e.g. Hegazy & Mayne 2002) that, in practice, such approach does not provide optimum results. A purely objective assessment (i.e. based exclusively on numerical criteria) is not feasible in practice, as at least the check on data quality and the definition of relevant parameters requires geotechnical background on the part of the user. Hence, in practice, homogeneity assessment should be both subjective and objective to some degree, and should include an efficient numerical analysis based on a suitable, expert choice of reference parameters.

A major issue to be addressed refers to whether the homogeneity should be assessed in terms of soil *composition* or soil *behaviour*. Research has shown that these do not display a one-to-one correspondence (e.g. Hegazy & Mayne 2002; Zhang & Tumay 2003). Soils which may be homogeneous in terms of composition may not be so in terms of mechanical behaviour. Hence, one should define and report the criteria adopted for physical homogeneity assessment. The in-situ penetration resistance of a compositionally homogeneous soil layer, for instance, increases with depth due to overburden stress and other in-situ conditions.

Most geotechnical testing methods provide direct information regarding the mechanical response of soil to penetration. Hence, homogeneity is usually assessed, in a quantitative perspective, on the basis of soil behaviour rather than composition. However, it may of interest to investigate variability in terms of the latter.

Hight & Leroueil (2003) provided the example of Bothkennar clay, for which three facies (mottled, bedded and laminated as shown in Figure 67; each with a distinctive fabric and associated with a distinct depositional environment) have been recognized despite a very uniform composition (as shown in Figure 68).

The influence of the fabric and depositional environment on the mechanical properties was observed experimentally (Figure 69). Hence, despite a compositional physical homogeneity, the geomaterial cannot be considered physically homogeneous from the point of view of mechanical behaviour. Variability may also occur due to man-induced factors. Hight & Leroueil (2003) showed the effects of different forms of damage and/or disturbance to samples from Bothkennar clay in terms of triaxial compression tests (Figure 70). Such variability in mechanical behaviour adds to the natural variability discussed earlier.

5.1 Assessment of physical homogeneity: literature review

In the following, a selection of methods making use of both subjective and objective criteria for the identification of physically homogeneous soil units is presented. The review in no way aims to be exhaustive.

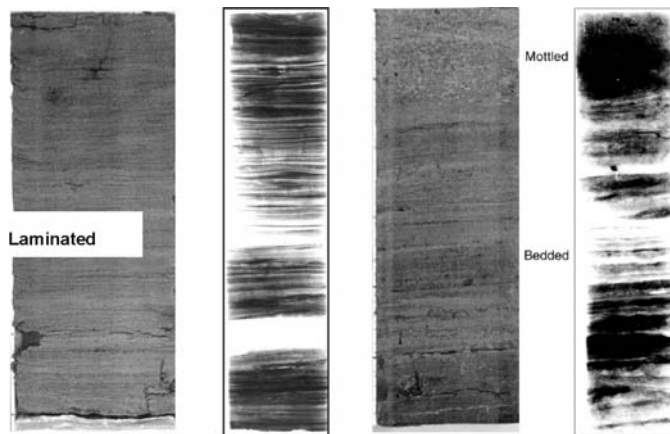


Figure 67. Facies of the Bothkennar clay (Hight & Leroueil 2003)

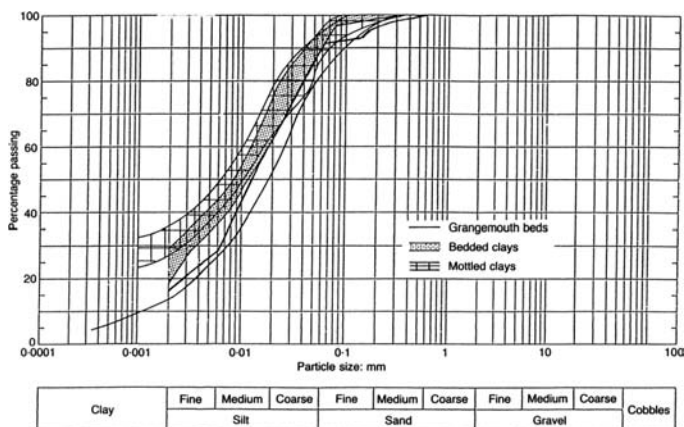


Figure 68. Grain size distribution of Bothkennar clay (Hight & Leroueil 2003).

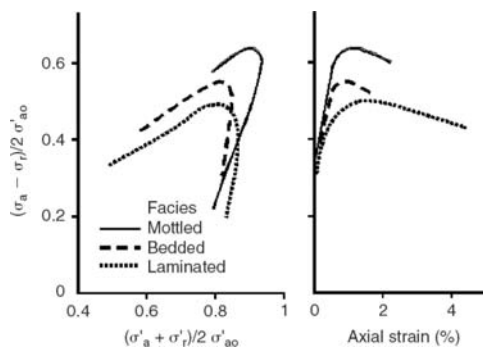


Figure 69. Non-homogeneity in mechanical behaviour of Bothkennar clay (Hight & Leroueil 2003).

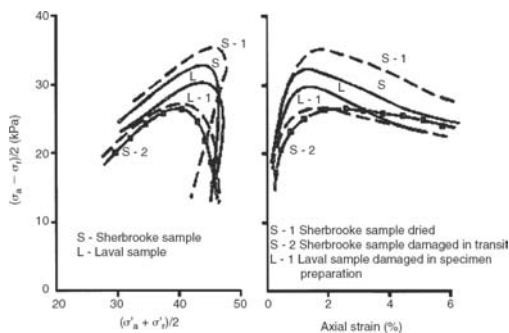


Figure 70. Effects of damage and/or disturbance to Bothkennar clay samples (Hight & Leroueil 2003).

5.1.1 Probabilistic-fuzzy stratigraphic delineation

Zhang & Tumay (2003) summarised the findings of their own previous research focusing on the CPT-based identification of physically homogeneous soils through a probabilistic and fuzzy methodology. The approach is based on the consideration that there is no univocal correspondence between existing

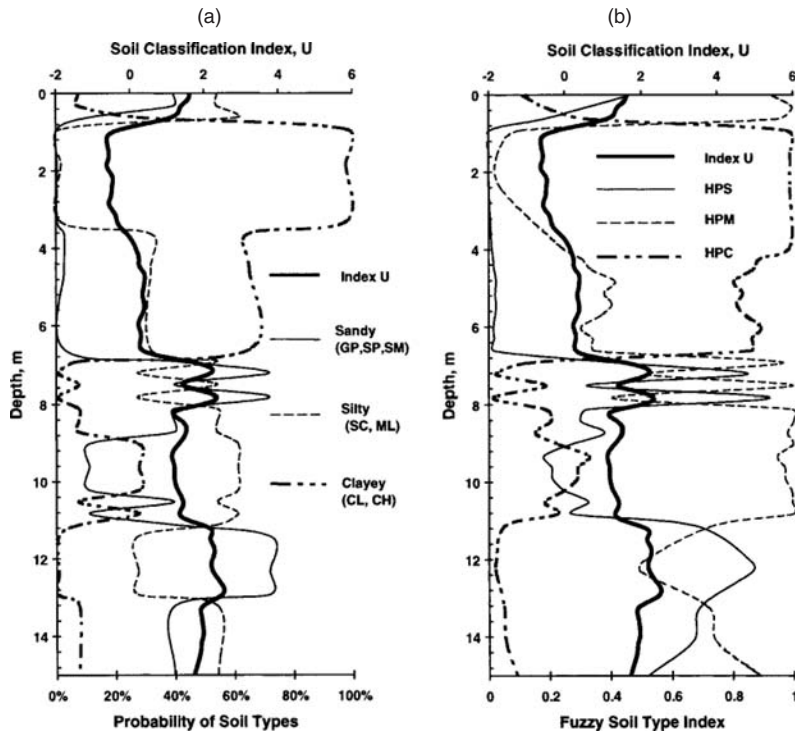


Figure 71. Identification of homogeneous units from soil classification for the NGES/Texas A&M site: (a) probability profile; (b) fuzzy soil type index profile (Zhang & Tumay 2003).

soil composition and the mechanical resistance to penetration, and tries to quantify the uncertainty in the composition-behaviour correspondence. The method is based upon a single soil classification index, obtained through a coordinate change procedure from existing CPT-based classification charts. At each depth where measurements are available, the method provides both a probabilistic and a fuzzy estimate of soil type in terms of behaviour. In the first case, the probability of a soil being “sandy”, “silty” or “clayey” is returned (with the sum of the three probabilities amounting to unity) on the basis of the soil classification index U . Fuzzy estimation provides a *degree of membership* (from 0: no membership to 1: full membership) of a soil datum (in this case U) to the fuzzy sets of “highly probably sandy” (HPS), “highly probably medium” (HPM) and “highly probably clayey” (HPC). The sum of membership values need not be unity.

Figure 71 (Zhang & Tumay 2003) illustrates the application of the method to data from the NGES/Texas A&M site. The approach can be integrated by the calculation of the intraclass correlation coefficient (e.g. Wickremesinghe & Campanella 1993; Cherubini et al. 2006) on the soil classification index to allow for the statistical identification of probable stratigraphic interfaces.

5.1.2 Statistical clustering

Hegazy & Mayne (2002) proposed a method for soil stratigraphy delineation by cluster analysis of piezocone data. The objectives of cluster analysis applied to CPT data were essentially: (a) objectively define similar groups in a soil profile; (b) delineate layer boundaries; and (c) allocate the lenses and outliers within sub-layers.

Visual inspection of data (e.g. OCR profile and index properties in Figure 72) may allow delineation of the main interfaces between homogeneous layers. Cluster analysis, on the contrary, allows identification of stratigraphic boundaries at various levels of resolution: if a configuration with a low number of clusters is referred to (e.g. 5 cluster-configuration in first column from left in Figure 73), the main boundaries are identified; with increasing number of clusters, greater resolution is achieved, by which less pronounced discontinuities are captured.

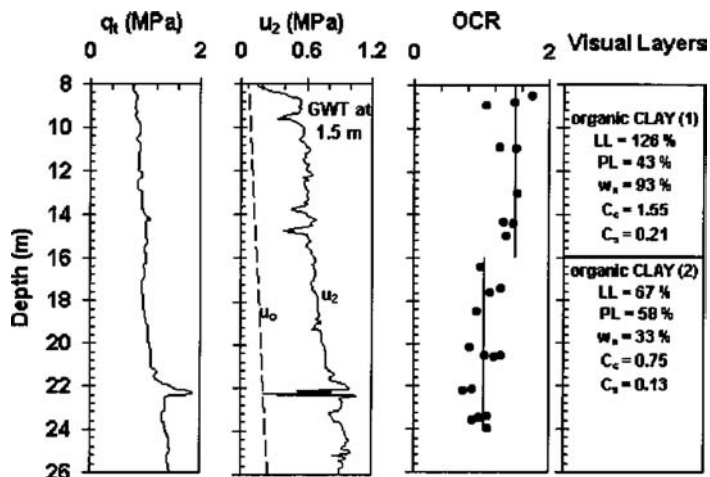


Figure 72. Identification of primary stratigraphic boundaries from cluster analysis – Recife, Brazil site (Hegazy & Mayne 2002).

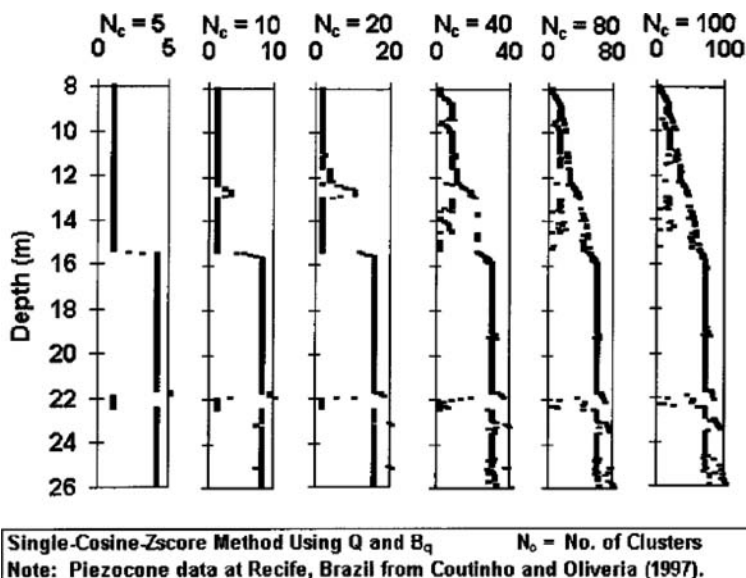


Figure 73. Identification of homogeneous soil units from clustering analysis: comparative configurations for variable number of clusters on same profile of corrected cone tip resistance – Recife, Brazil site (Hegazy & Mayne 2002).

Hegazy & Mayne (2002) and Facciorusso & Uzielli (2004) assessed the very good capabilities of clustering techniques for CPT-based soil stratigraphy delineation. However, they also observed that clustering is computationally expensive, and is difficult to implement for large data sets.

5.1.3 Modified Bartlett's statistic method

Phoon et al. (2003c) proposed a statistical moving window methodology for the identification of physically homogeneous soil units using Bartlett's statistic. The sampling window is divided into two equal

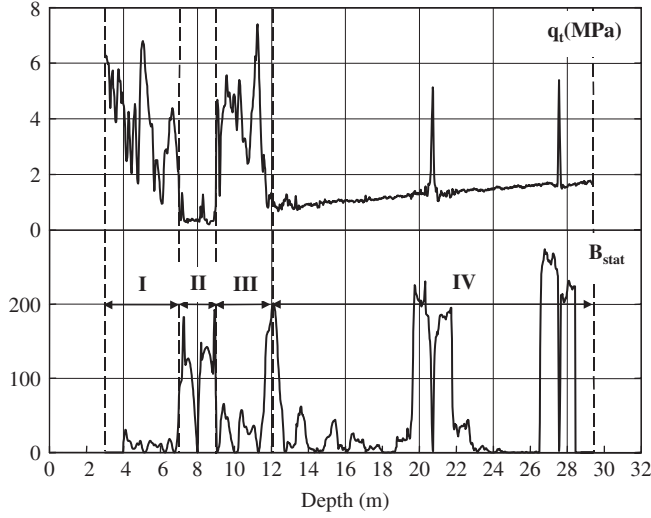


Figure 74. Example identification of potentially homogeneous soil units I–IV based on Bartlett statistic profile (bottom) from corrected cone tip resistance data (top) (Phoon et al. 2004).

segments and the sample variance is calculated from data points lying within each segment. For the case of two sample variances, σ_{B1}^2 and σ_{B2}^2 , the Bartlett test statistic reduces to

$$B_{\text{stat}} = \frac{2.30259(m_B - 1)}{C_B} \left[2 \log \sigma_B^2 - (\log \sigma_{B1}^2 + \log \sigma_{B2}^2) \right] \quad (43)$$

where m_B is the number of data points used to evaluate σ_{B1}^2 and σ_{B2}^2 . The total variance, σ_B^2 , is defined as

$$\sigma_B^2 = \frac{\sigma_{B1}^2 + \sigma_{B2}^2}{2} \quad (44)$$

The constant C_B is given by

$$C_B = 1 + \frac{1}{2(m_B - 1)} \quad (45)$$

Bartlett's statistic indicates the difference between the sample variances σ_{B1}^2 and σ_{B2}^2 in the two adjacent segments. The value of the Bartlett statistic increases with increasingly different sample variances, and is equal to zero if the sample variances are equal. A continuous Bartlett statistic profile can be generated by moving a sampling window over a spatially ordered data profile. Peaks in the Bartlett statistic profile indicate possible interfaces between physically homogeneous soil units.

Figure 74 (Phoon et al. 2004) shows an application of the methodology to corrected cone tip resistance data. Phoon et al. (2003c) stated that the identification process based on the modified Bartlett statistic profile is only a preliminary assessment, and that engineering judgement should be exercised to ensure that results are physically meaningful. Hence, the Bartlett statistic profile is to be interpreted only in a qualitative way in the context of soil boundary identification.

5.1.4 Statistical moving window

Uzielli (2004) proposed a statistical moving window procedure to identify physically homogeneous soil units making use of coefficients of variation of selected properties. Each moving window is made up of two semi-windows of equal height above and below a centre point. A suitable height of the moving window of $W_d = 1.50$ m was established by Uzielli (2004) on the basis of past research (e.g. Lunne et al. 1997) and calibration of results with available borehole logs. At each centre point (identified by its depth z_c), the mean, standard deviation and coefficient of variation (COV) were calculated for data lying in the interval $z_c - W_d/2 \leq z \leq z_c + W_d/2$, corresponding to the upper and lower limits of the moving window.

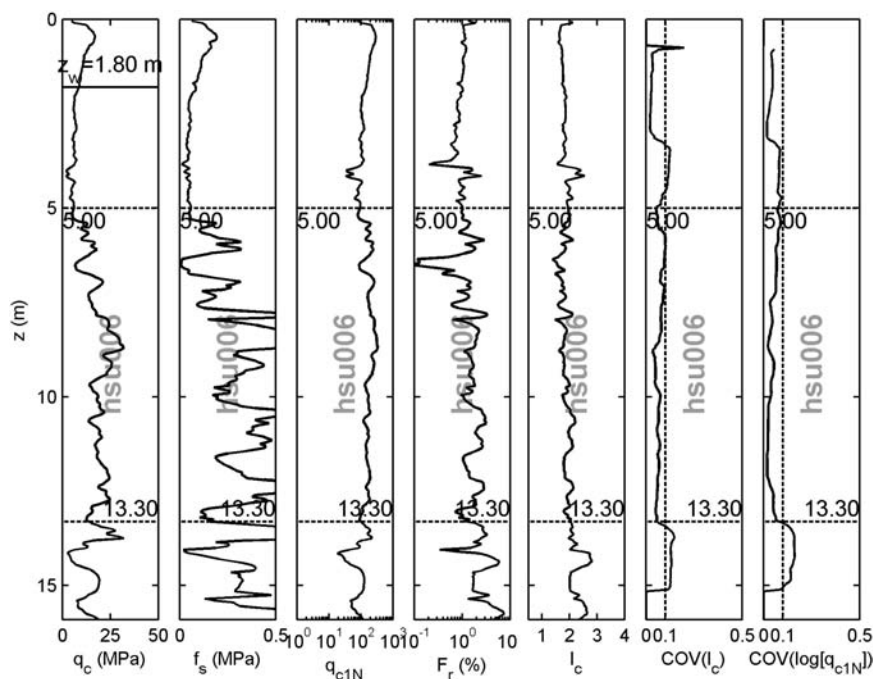


Figure 75. Application of the moving window procedure to cone penetration testing data (Uzielli 2004).

HSUs are essentially identified by delineating soundings into sections where the COV of one or more user-selected parameters are less than a preset threshold. Harr (1987) proposed a value of 0.1 for COV as the upper limit for “low dispersion” in soil properties. A minimum width for homogeneous units can be preset by the user.

Uzielli et al. (2005a) applied the procedure to CPT soundings using the normalised, dimensionless cone penetration resistance q_{c1N} , the normalised friction ratio F_R and the soil behaviour classification index, I_c . These parameters have been referred to in previous sections of the paper.

Figure 75 illustrates the application of the method to cone penetration testing data. The coefficients of variation of the logarithm of normalised cone tip resistance and soil behaviour classification index are plotted in the two rightmost diagrams. The spatial extension in which they are both smaller than 0.10 (for at least 4.50 m consecutively) is identified as a physically homogeneous soil unit.

In principle, the procedure can be applied to data from other in-situ tests, provided that a parameter which is related to soil classification is available. In the case of flat dilatometer testing (DMT), for instance, the material index I_D is a suitable parameter. Figure 76 shows an application of the method to DMT data, where the coefficient of variation of I_D is the reference parameter.

5.2 Second-moment representation of homogeneous soil units

The distribution of values of a soil property in a physically homogeneous soil unit can be represented concisely in a second-moment sense by a mean and a standard deviation. In Figure 77 (Uzielli et al. 2005a), normalised friction ratio and normalised cone tip resistance data from 70 physically homogeneous soil units are plotted on Robertson's F_R - q_{c1N} chart through their second-moment statistics: each point represents one unit, with the circles located at the mean values and error bars indicating plus and minus one standard deviation.

5.3 Assessment of identification methods

The performance of the homogeneous soil unit identification methods should be assessed using geotechnical knowledge or statistical criteria. A variety of soil classification charts based on in-situ tests are available in the geotechnical literature; these provide an effective means for qualitative assessment of

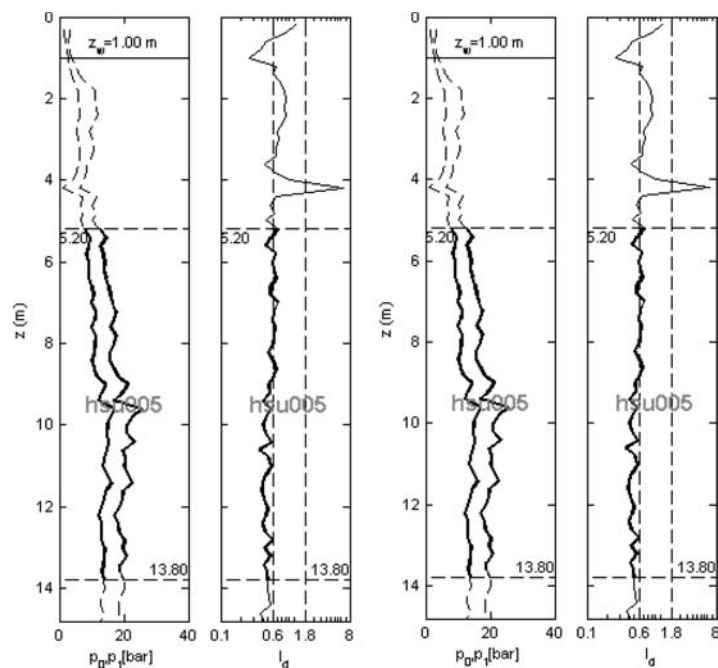


Figure 76. Application of the moving window procedure to flat dilatometer testing data.

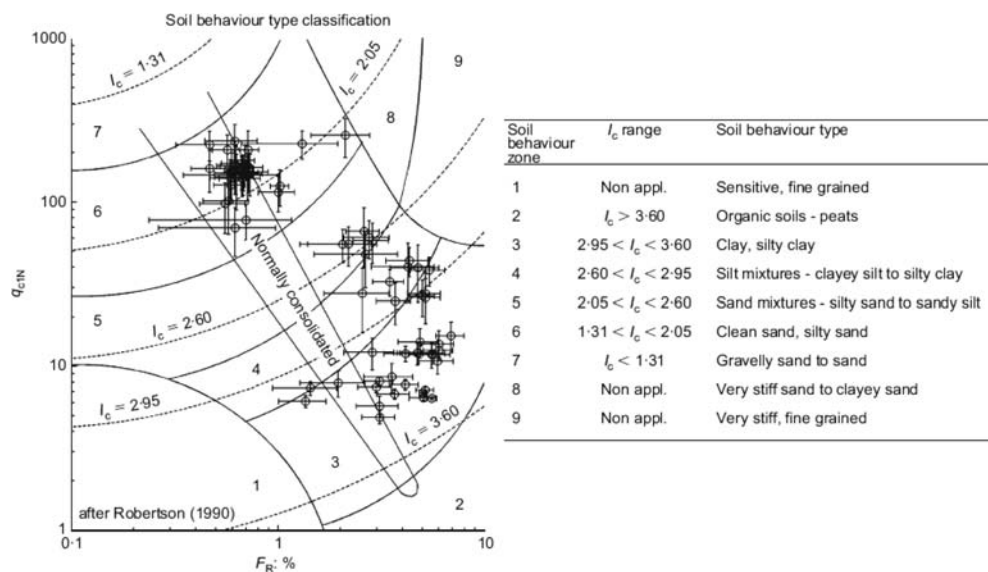


Figure 77. Representation of second-moment data from 70 physically homogeneous soil units on Robertson's CPT-based F_R - q_{c1N} chart (Uzielli et al. 2005a).

the homogeneity of a data set. If data points from a soil unit plot as a well-defined cluster, homogeneity can be expected. For CPTU testing, for instance, Robertson's (1990) classification charts were used by Phoon et al. (2004), Hegazy & Mayne (2002) and Uzielli et al. (2005a) to assess the performance of the methods they proposed (see Figure 78, Figure 79 and Figure 80). For DMT testing, Marchetti & Crapps' (1981) soil classification chart can be used (e.g. Figure 81).

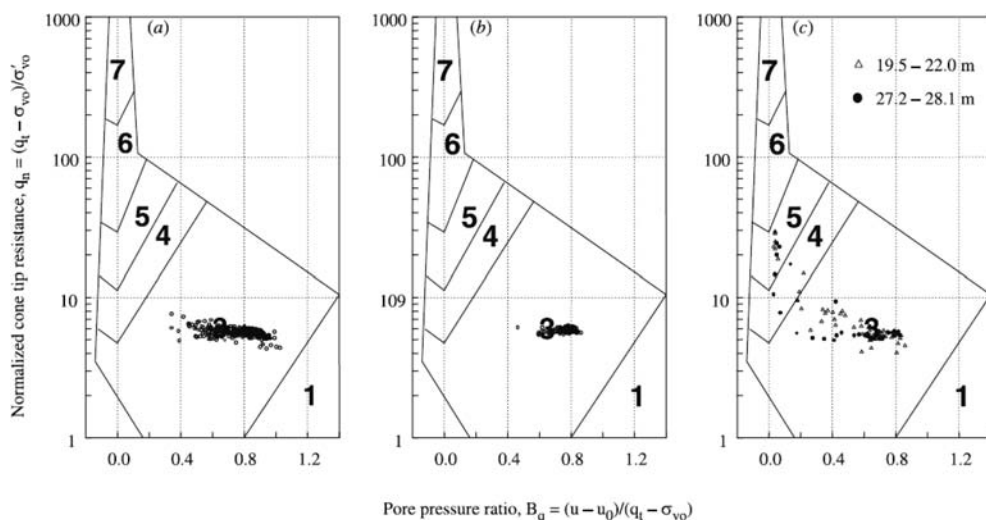


Figure 78. Assessment of the Bartlett statistic method: plotting data from a physically homogeneous soil unit on Robertson's CPTU-based B_q - q_n chart (Phoon et al. 2004).

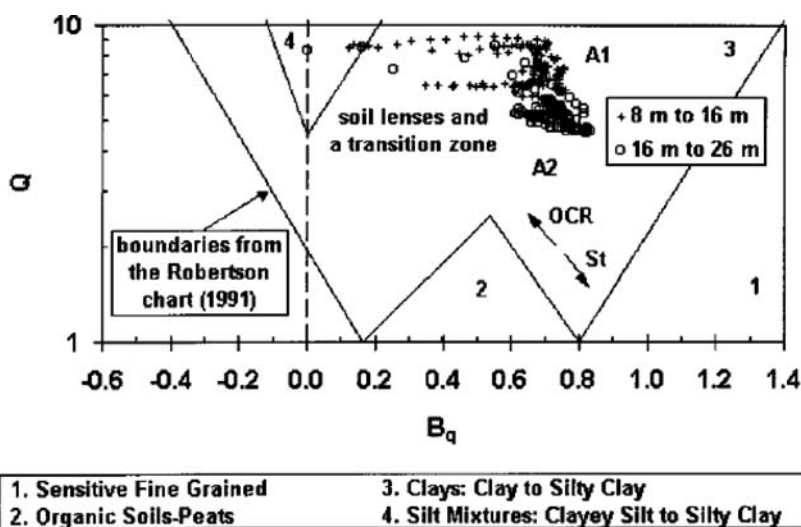


Figure 79. Assessment of statistical clustering method: plotting data from a physically homogeneous soil unit on Robertson's CPTU-based B_q - Q chart (Hegazy & Mayne 2002).

Uzielli et al. (2005a) also employed a statistical criterion to assess the performance of the moving window statistical methodology presented in §5.1.4. They calculated the coefficient of variation of the soil behaviour classification index I_c (which is an efficient descriptor of soil behaviour) in each identified homogeneous soil unit. A low coefficient of variation indicates low data dispersion and, hence, homogeneity. The procedure was shown to perform very well both in cohesive- and non-cohesive-behaviour soil units, as the average of the coefficients of variation was 0.02, with values from individual soil units never exceeding 0.10 (a value indicating low dispersion according to Harr's rule of thumb).

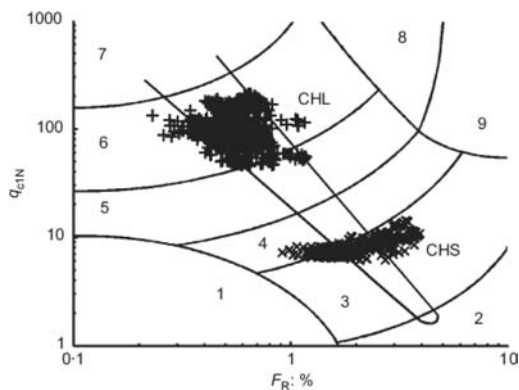


Figure 80. Assessment of the statistical moving window method: plotting data from two physically homogeneous soil units (CHS: cohesive-behaviour; CHL: cohesionless behaviour) on Robertson's CPT-based F_R - q_{c1N} chart (Uzielli et al. 2005a).

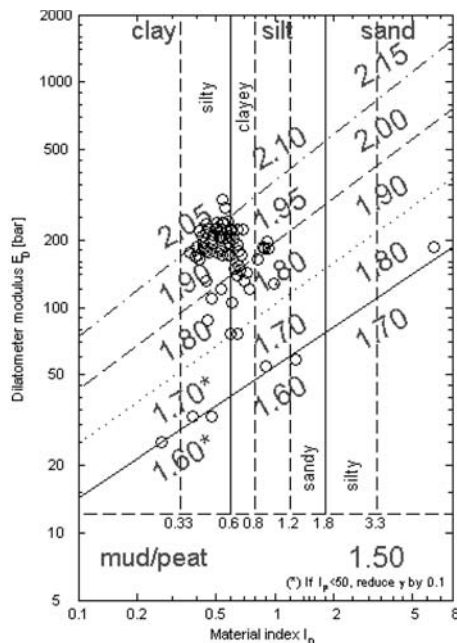


Figure 81. Assessment of the statistical moving window method: plotting data from a physically homogeneous soil unit on Marchetti and Crapps' (1981) I_D - E_D chart.

6 ASSESSMENT OF WEAK STATIONARITY

Stationarity is an important prerequisite for stochastic modelling of soil properties because the statistical procedures employed are based on the assumption that data samples consist of stationary observations.

Among stochastic processes, *strictly stationary* processes are characterised by the fact that *all* finite-dimensional probability distributions are invariant to translation. As a consequence, all of their statistical properties (e.g. mean, variance, skewness, kurtosis, etc.) are spatially invariant. Strict stationarity is a severe requirement in practice and mostly of conceptual importance only. In practice, it is necessary and convenient to relax the constraint of strict stationarity and refer to *weak stationarity*. A process is said to be stationary in the second-order sense (or *weakly stationary*) if: a) its mean is constant (i.e. there are no trends in the data); b) its variance is constant; and c) the correlation between the values at any two points at distinct spatial locations, depends only on the interval in the index set between the points, and not on the specific spatial location of the points themselves, i.e. autocorrelation is a function of separation distance only. In the case of laboratory or in-situ geotechnical testing, stationarity can only be identified in a weak or second-order sense because of the limitations in sample size.

6.1 Testing for weak stationarity

There are two general approaches to statistical testing for stationarity; *parametric* and *nonparametric*. Parametric approaches require the formulation of hypotheses regarding the nature and distribution of the data set under investigation. Non-parametric approaches, on the contrary, do not make any basic assumptions about the nature of the system, e.g. they are not based on the knowledge or assumption that the population is normally distributed. By making no assumptions about the nature of the data, non-parametric tests are more widely applicable than parametric tests which often require normality in the data. While more widely applicable, the trade-off is that non-parametric tests are less powerful than parametric tests. As stationarity is tested by the use of statistical procedures, it is perhaps useful to remark that assessment is referred to a specified *confidence level*.

It should be emphasised (e.g. Baecher & Christian 2003) that stationarity is an assumption of the model, and may only approximate the real-world situation. Also, stationarity usually depends upon

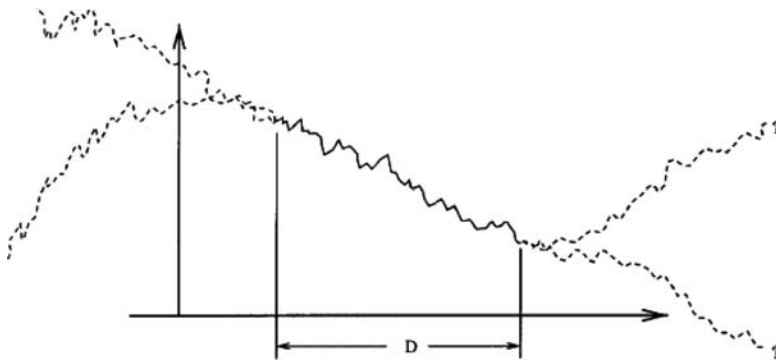


Figure 82. Dependency of weak stationarity from scale of investigation (Fenton 1999a).

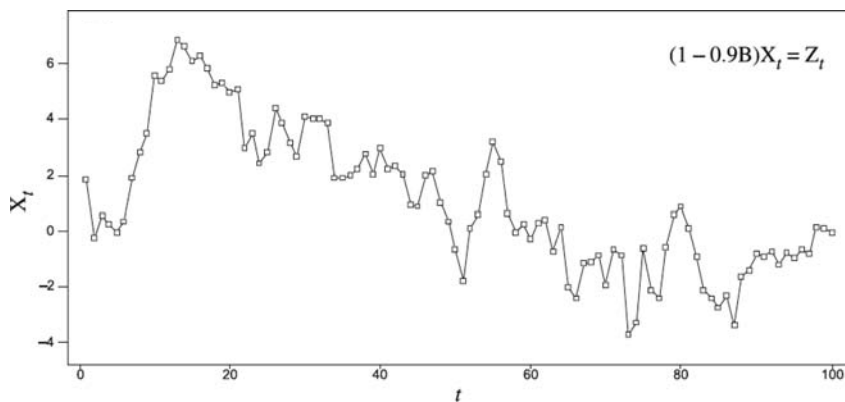


Figure 83. Sample realisation with apparent non-stationarity in the mean (Phoon et al. 2004).

scale. Phoon et al. (2003c), recalling the fractal behaviour of soil properties, specified that even weak stationarity cannot be verified in a strict sense over a finite length because longer scale fluctuations could be mistakenly identified as a non-stationary component (e.g. trend). Hence, at a small scale such as a construction site, soil properties may behave as if drawn from a stationary process; whereas, the same properties over a larger region may not be.

The dependence of weak stationarity from the scale of investigation is shown in Figure 82 (Fenton 1999a), in which the process shows a clear trend over the spatial extension D (and is therefore non-stationary). However, if the scale of investigation is increased, the process could be part of a larger-scale stationary process (the fluctuating line) or of another non-stationary process (the quasi-monotonic line with a negative trend). Figure 83 (Phoon et al. 2004) shows a realisation displaying an apparent non-stationarity in the mean (i.e. a spatial trend) though it was simulated from a stationary model.

In the following, two stationarity assessment procedures are proposed. The first procedure is referred to as *Kendall's tau test*. Among the classical statistical tests which have been used in the geotechnical literature, Kendall's tau test has been identified (e.g. Jaksa 1995) as the most powerful for geotechnical applications, though recognising that, in some instances, the test failed to reject apparently non-stationary data for moderately small samples. Kendall's tau test (e.g. Daniel 1990) is a non-parametric test for statistical independence. As statistical independence implies stationarity (the converse is not true), Kendall's test can be used for assessing the latter.

Kendall's test, as other classical stationarity tests (e.g. statistical runs test, Spearman's rank coefficient), is based on the assumption of spatially uncorrelated input data. This assumption is antithetical to the fundamental premise in geotechnical variability analysis, namely the existence of a correlation structure in spatially varying soil properties. Due to the resulting bias in autocorrelation coefficients in the correlated case, the application of classical statistical tests may result in unconservative assessments, i.e. non-stationary sets may be erroneously classified as weakly stationary.

Note that Kendall's test (or other classical parametric or non-parametric tests) is not strictly applicable to correlated data. To the writers' knowledge, the only non-parametric method that is developed for correlated data is the boot-strapping method presented by Phoon & Fenton (2004) and Phoon (2006a). It is difficult to develop a non-parametric method for correlated data because minimal assumptions are imposed not merely on probability distribution, but on the correlation structure as well. In contrast, classical non-parametric methods for independent data only minimize assumptions on probability distributions.

The procedures reported in the following implicitly neglect measurement and random testing errors, i.e. they assume that measured values reflect the true values of the parameter of interest. This assumption is acceptable for tests which have been shown to be largely operator-independent and to have very low random measurement errors such as the CPT or DMT. Results of tests with high measurement and random uncertainty (such as the SPT, which also has a very large measurement interval) are not reliable inputs for the method.

6.1.1 Kendall's tau test

Kendall's test involves the calculation of the test statistic, τ_{ken} , which measures the probability of concordance minus the probability of discordance between measurements in a data set. The test statistic is given by:

$$\tau_{ken} = \frac{S_{ken}}{\frac{1}{2}n_d(n_d - 1)} \quad (46)$$

where n_d is the number of pairs of observations in a data set; and

$$S_{ken} = P_{ken} - Q_{ken} \quad (47)$$

in which P_{ken} is the number of concordant pairs of observations and Q_{ken} is the number of discordant pairs of observations. To calculate such parameters, a total of $(1/2)n_d(n_d - 1)$ comparisons are made between all possible pairs of observations. A pair of observations (x_i, y_i) , (x_j, y_j) is said to be concordant if $x_i > x_j$ and $y_i > y_j$, while it is said to be discordant if $x_i > x_j$ and $y_i < y_j$. A visual representation of concordant and discordant data pairs is provided in Figure 84.

The values of τ_{ken} range from -1 to $+1$, indicating, respectively, perfect negative and positive correlation. A value close to zero indicates low correlation.

For $n_d \leq 40$, critical values of τ_{ken} for rejecting the null hypothesis of independence are available in tabulated form (e.g. Daniel 1990). For $n_d > 40$, the following statistic is calculated from τ_{ken} :

$$z_{\tau,ken} = \frac{3\tau_{ken}\sqrt{n_d(n_d - 1)}}{\sqrt{2(2n_d + 5)}} \quad (48)$$

The statistic $z_{\tau,ken}$ is normally distributed with zero mean and unit standard deviation; hence, it can be compared to values in standard normal distribution tables. For a 95% confidence level, for instance, if $|z_{\tau,ken}| \leq 1.96$ the data can be assumed to be statistically independent.

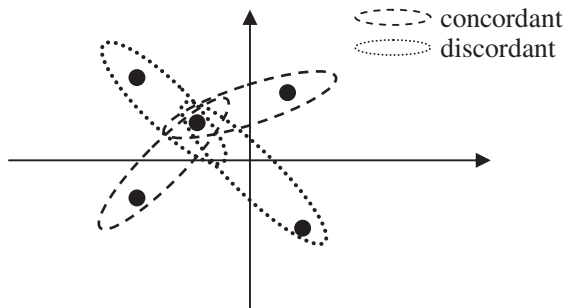


Figure 84. Concordant and discordant pairs of observations.

6.1.2 The MBSR method

A procedure based on Bartlett's statistic was first proposed by Phoon et al. (2003c) with the aim of providing a more rational basis for rejecting the null hypothesis of stationarity in the correlated case as it incorporates the correlation structure in the underlying data and as it includes all fundamental phases of random field modelling analysis (stationarity, choice of trend function and autocorrelation model).

The method was subsequently integrated by restrictive conditions on the fit of autocorrelation models by Uzielli (2004); the resulting procedure is referred to hereinafter as MBSR (i.e. Modified Bartlett Statistic Revised).

MBSR requires the preliminary calculation of the Bartlett statistic profile (see §5.1.3), the estimation of the scale of fluctuation by fitting theoretical autocorrelation models (see §4.3.1) and the subsequent comparison between the maximum value of the Bartlett statistic and a critical value. The latter is calculated as follows. First, the *profile factors* in Table 26 are defined. In Table 26, δ is the scale of fluctuation; Δz is the measurement interval; L_d is the spatial length of the soil record of length; and W_B is the spatial extension of half the sampling window.

The number of points in one scale of fluctuation, k_B , should lie between 5 and 50. The normalised sampling length, I_1 , is also taken to lie between 5 and 50. Phoon et al. (2003c) suggested that the normalised segment length be chosen as $I_2 = 1$ (for $k_B \geq 10$) and $I_2 = 2$ (for $k_B < 10$).

Once the Bartlett statistic profile is obtained, the peak value $B_{\max}$ is identified. Critical values (see Table 27) of this modified Bartlett test statistic at 5% level of significance for several autocorrelation models, among which single exponential (SNX), cosine exponential (CSX), second-order Markov (SMK) and squared exponential (SQX), were provided by Phoon et al. (2003c) using the spectral approach by Shinozuka & Deodatis (1991). The null hypothesis of stationarity in the variance is rejected at 5% level of significance if $B_{\max} > B_{\text{crit}}$.

Phoon et al. (2003c) observed that the stationarity rejection criterion is significantly dependent on the ACM employed. The criterion is most strict for SNX and becomes increasingly relaxed in the order of CSX, SMK and SQX.

Following application of the MBSR method, three outcomes are possible:

1. MBSR is not applicable in its entirety, due to one or more of the following: a) the minimum value of the determination coefficient $R^2_{\min}$ is not reached; b) one or more of the dimensionless profile factors of the MBS procedure (k_B , I_1 , I_2), which depend on the scale of fluctuation, δ , sample spacing, Δz , and sample size, n_B , falls outside the ranges established by Phoon et al. (2003c);
2. MBSR can be applied, but $B_{\max} \geq B_{\text{crit}}$; thus, the profile is classified as non-stationary at 5% significance level;
3. MBSR procedure can be applied in its entirety, and at least one ACM provided: a) stationarity at the 5% level; and b) a coefficient of determination $R^2 \geq R^2_{\min}$. If more than one ACM satisfies the above conditions, the scale of fluctuation deriving from the ACM with the maximum R^2 is adopted.

Table 26. Profile factors for the Bartlett statistic-based method (Phoon et al. 2003c).

Profile factor	Expression
Number of points in one scale of fluctuation	$k_B = \delta / \Delta z$
Normalised sampling length	$I_1 = L_d / \delta$
Normalised segment length	$I_2 = W_B / \delta$

Table 27. Critical modified Bartlett test statistics at 5% level of significance for SNX, CSX, SMK and SQX autocorrelation models (from Phoon et al. 2003c).

ACM	I_2	
SNX	1	$B_{\text{crit}} = (0.23k_B + 0.71) \ln(I_1) + 0.91k_B + 0.23$
SNX	2	$B_{\text{crit}} = (0.36k_B + 0.66) \ln(I_1) + 1.31k_B - 1.77$
CSX	1	$B_{\text{crit}} = (0.28k_B + 0.43) \ln(I_1) + 1.29k_B - 0.40$
SMK	1	$B_{\text{crit}} = (0.42k_B - 0.07) \ln(I_1) + 2.04k_B - 3.32$
SQX	1	$B_{\text{crit}} = (0.73k_B - 0.98) \ln(I_1) + 2.35k_B - 2.45$

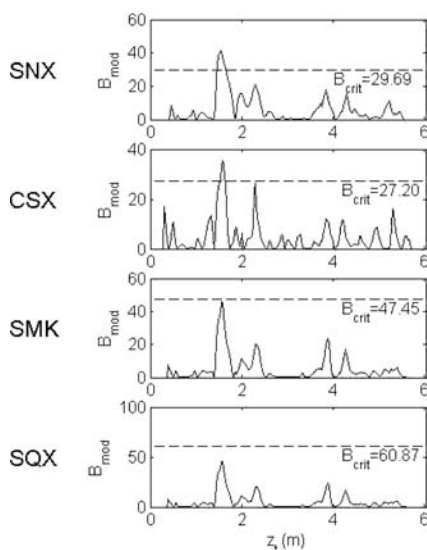


Figure 85. Output of application of MBSR to a normalised cone tip resistance profile (Uzielli 2004).

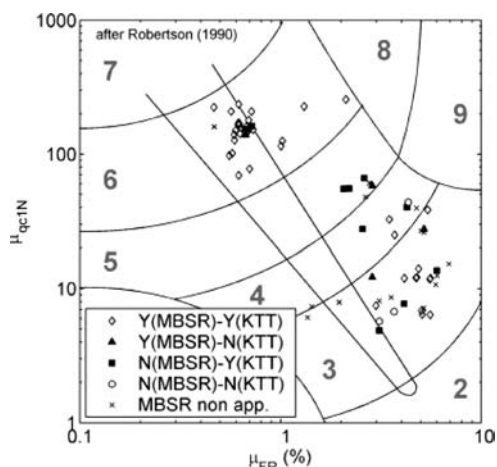


Figure 86. Comparative assessment of weak stationarity (Y) or non-stationarity (N) by MBSR versus KTT (Uzielli et al. 2004).

Thus, a given value of δ_v calculated from one of the ACMs is accepted only if: a) it refers to a weakly stationary fluctuating component [i.e. $B_{\max} < B_{\text{crit}}$ in the MBSR procedure]; b) the determination coefficient, R^2 , of the fit of the ACM to the significant portion of the autocorrelation function (defined by the parameter r_B) does not fall below a user-defined threshold.

An example of MBSR-based weak stationarity assessment for a profile of normalised cone tip resistance in a homogeneous soil unit is shown in Figure 85.

6.1.3 Comparison with results of Kendall's tau test for statistical independence

To verify the severity of the MBSR test compared to a statistical test which assumes uncorrelated input data, a comparative assessment of stationarity with Kendall's tau test at 95% confidence level was performed by Uzielli et al. (2004) on detrended profiles of normalised cone tip resistance. Figure 86 compares stationarity assessment using MBSR and KTT. Both tests agree in the majority of the cases (open diamond and circle). However, HSUs that are MBSR stationary are rarely not KTT-stationary (solid triangles). Conversely, there are quite a number of HSUs that are KTT-stationary, but not MBSR-stationary (solid squares). If one assumes that MBSR provides more accurate stationarity assessments, the power of KTT (rejecting the null when it is false) is lower, i.e. KTT is less discriminatory. However, the performance of the tests is comparable. On the basis of such findings, it would appear that Kendall's test can be used with sufficient confidence. More investigations are warranted to support these conclusions.

7 ADVANCED MODELLING OF SPATIALLY RANDOM GEOTECHNICAL SYSTEMS

The description of a random field in the second-moment sense through a mean, standard deviation, a scale of fluctuation and a spatial correlation function is useful to characterise a spatially variable soil property. Moreover, it allows the spatial averaging effect to be investigated, which significantly affects quantitative variability assessment.

However, if the results of random field modelling are to be used in reliability-based engineering applications, some possible limitations in this approach should be recognised.

For instance, it could be suspected that, if spatial variability of soil properties is included in an engineering model, stresses and/or displacements which would not appear in the homogeneous case (i.e. in which variability is not addressed) could be present. Hence, addressing spatial variability may allow a more realistic modelling of the *behaviour* of geotechnical systems.

One of the most important benefits of random field modelling is the capacity to simulate data series. By using sets of random field simulations and implementing the variability in non-linear finite element meshes, the Monte Carlo technique can be used to predict reliability of geotechnical systems with spatially variable properties.

A number of studies have focused, in recent years, on the combined utilisation of random fields, non-linear finite element analysis and Monte Carlo simulation for investigating the behaviour and reliability of geotechnical systems when the variability of soil properties which are relevant to the main presumable failure mechanisms is considered. A number of these are reviewed in the following.

Though the aforementioned studies address a wide variety of geotechnical systems, a number of important general observations can be made on the basis of their results. First, when soils are modelled as spatially variable, the modelled failure mechanisms are quite different – and significantly more complex – than in the case of deterministic soil properties. One example is that a footing resting on a spatially varying medium will fail realistically on one side, in contrast to the classical symmetrical Prandtl bearing capacity failure pattern. Another simple example is that realistic local slips along a long embankment can be reproduced. None of these behaviours can be simulated by FEM with conventional homogeneous or stratified soil profiles. More in-depth examples are given by Kim (2005) and Kim & Santamarina (2006). Second, there generally exists a *critical correlation distance* which corresponds to minimum reliability. Third, phenomena governed by highly non-linear constitutive laws are affected the most by spatial variations in soil properties.

Hence, variance reduction alone is unable to convey a comprehensive picture of the implications of spatial variability on the behaviour of a geotechnical system. *Both* statistical variability (i.e. in second-moment sense) and spatial variability (i.e. spatial correlation) of soil properties affect the reliability of geotechnical systems. Three-dimensional aspects are also relevant in some cases.

The number of studies making use of random field simulation, finite elements and Monte Carlo simulation is still limited. Important assumptions such as that of anisotropy in spatial correlation structures are not always addressed, and the selection of values for probabilistic descriptors of the random fields are often assumed from literature. Moreover, such approaches are not conducted, at present, at a routine level. Nonetheless, the importance of the results they are suggesting should serve as a stimulus for future research and for the transposition of results to a more reproducible and user-friendly perspective.

7.1 Slope stability

Griffiths & Fenton (2004) proposed the application of the Random Finite Element Method (RFEM) to slope stability analysis. The RFEM (e.g. Griffiths & Fenton 1998) accounts for spatial correlation and spatial averaging effects, and, in contrast to probabilistic approaches such as those described in §4.9, does not require *a priori* assumptions regarding the shape or location of the failure mechanism. Hence, the failure mechanism is able to seek its way through the modelled soil volume: this is significantly more consistent with the physical reality of a spatially variable soil. Figure 87 shows two possible spatial distributions of normalised strength corresponding to two values of the normalised scale of fluctuation (which provides a measure of the magnitude of spatial correlation of normalised strength) for the simple model slope referred to in §2.17 and §4.9. Darker areas correspond to higher normalised undrained shear strength. It is unlikely, for instance, that the mesh with weaker spatial correlation (top) may develop a slip surface similar to the configuration with stronger spatial correlation (bottom). RFEM analysis is able to capture the diversity between the modes of failure of weakly and strongly spatially correlated profiles (top and bottom, respectively, of Figure 88).

Figure 89 shows the probability of failure versus the coefficient of variation of the lognormal random field for various values of the normalised scale of fluctuation. The existence of a crossover point, which indicates a change in the relationship between coefficient of variation, probability of failure and factor of safety, may be observed.

In comparison with the results of the spatially averaged Monte Carlo simulation described in §4.9, and for any given mean of the lognormal random field, RFEM yields lower probabilities of failure for low coefficients of variation and higher probabilities of failure for higher coefficients of variation. This is shown in Figure 90.

In synthesis, Griffiths & Fenton (2004) stated that simplified probabilistic analyses, in which spatial variability is not accounted for, could lead to unconservative estimates of reliability. This effect is most evident at lower factors of safety or for random fields of undrained strength with high coefficients of variation.

7.2 Seepage through water retaining structures

Griffiths & Fenton (1998) investigated the influence of the variability in soil permeability on the second-moment statistics of the exit gradient at the downstream side of a water-retaining earth structure in both two and three dimensions. Figure 91 illustrates the difference in geometry in a typical flow net between

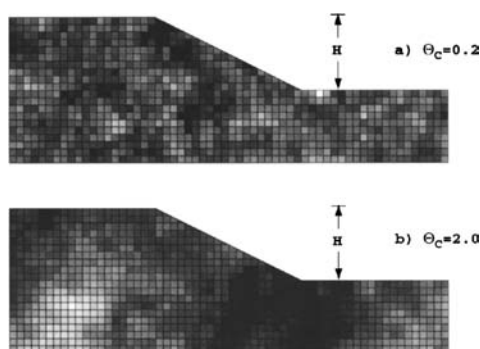


Figure 87. Influence of normalised scale of fluctuation on spatial variation of soil properties for RFEM analysis (Griffiths & Fenton 2004).

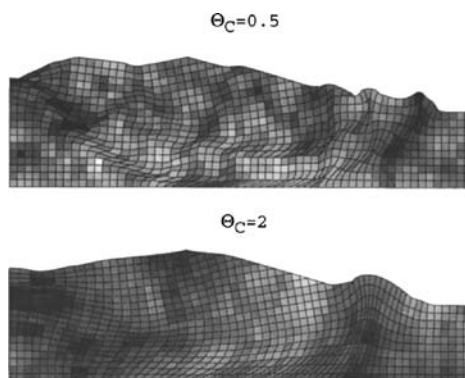


Figure 88. Examples of failure modes for weakly (top) and strongly (bottom) spatially correlated configurations of the model slope using RFEM (Griffiths & Fenton 2004).

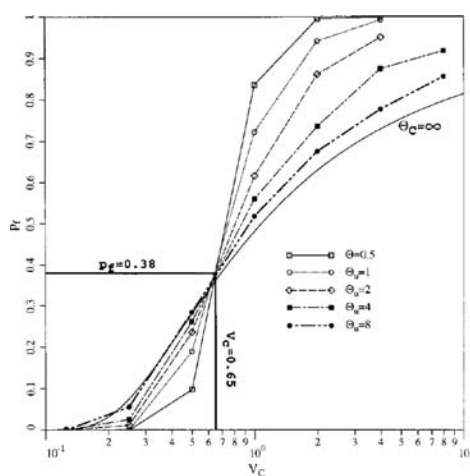


Figure 89. Probability of failure versus coefficient of variation for various normalised scales of fluctuation: results of RFEM analysis (Griffiths & Fenton 2004).

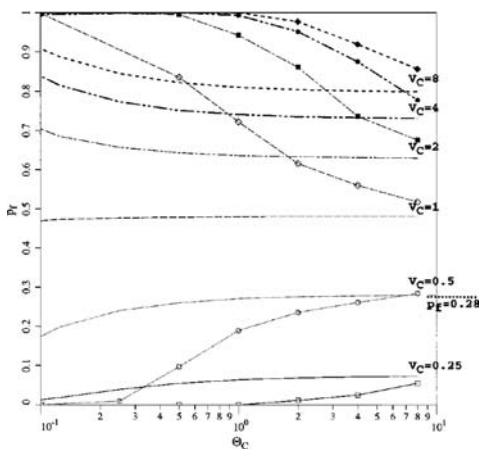


Figure 90. Comparison of the probability of failure for various coefficients of variation of normalised undrained strength predicted by RFEM (curves with points) and by probabilistic analyses accounting for spatial averaging (Griffiths & Fenton 2004).



Figure 91. Typical geometry of flow nets for: (a) deterministic soil permeability; and (b) spatially random soil permeability (Griffiths & Fenton 1998).

(a) the deterministic permeability; and (b) spatially variable permeability cases. A lognormal distribution was selected for the random field of permeability.

The results of the two-dimensional analysis are illustrated in Figure 92: (a) shows the mean value of exit gradient versus the coefficient of variation for different values of the scale of fluctuation; (b) shows the mean value of exit gradient versus the scale of fluctuation for various coefficients of variation; (c) shows the standard deviation of exit gradient versus the coefficient of variation for different values of

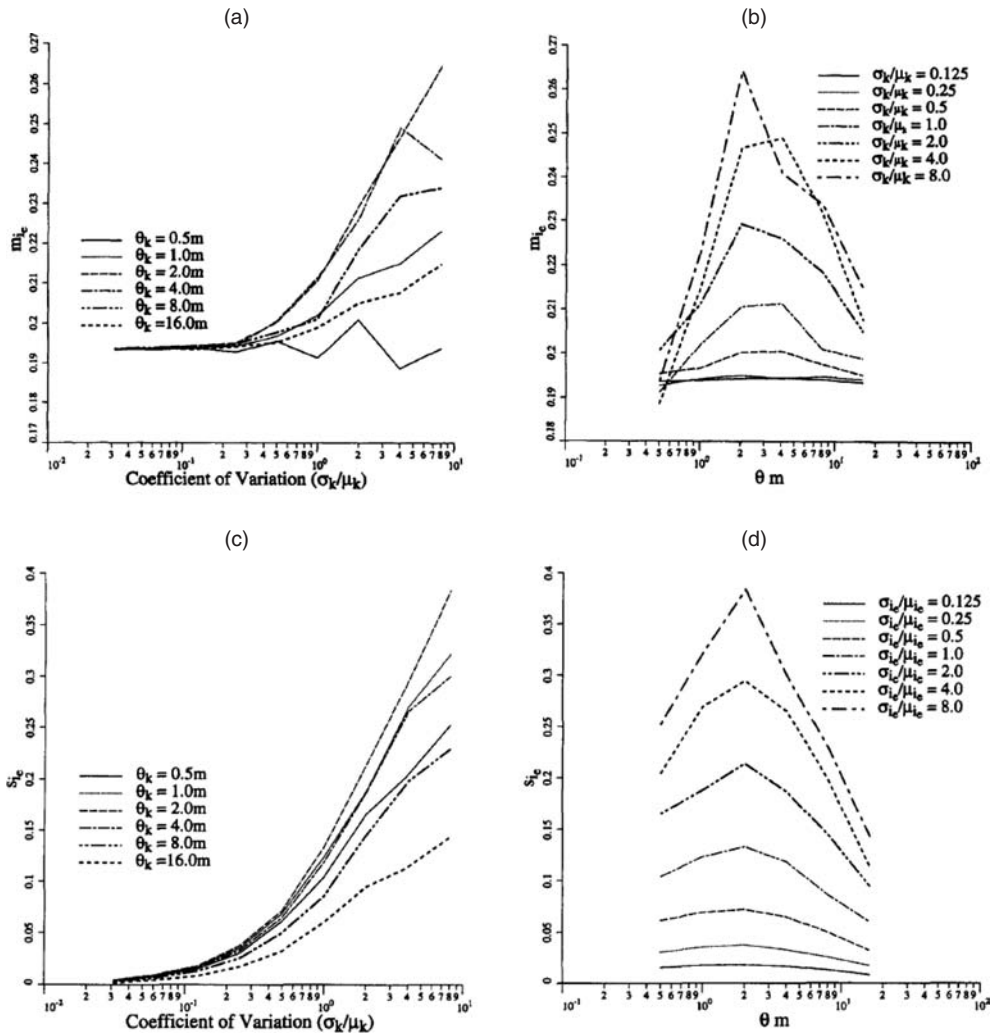


Figure 92. Results of 2D analysis of impact of variability in permeability on exit gradient from water retaining earth structures (Griffiths & Fenton 1998).

the scale of fluctuation; (b) shows the standard deviation of exit gradient versus the scale of fluctuation for various coefficients of variation.

The corresponding plots for the three-dimensional case are shown in Figure 93. In both cases, Griffiths & Fenton (1998) inferred that the inclusion of variability in permeability provided results that were significantly different from the deterministic case. First, the mean gradient remains essentially constant around the deterministic value for small scales of fluctuation, but tends to increase for larger scales of fluctuation (see Figure 92a for the 2D case; Figure 93a for the 3D case). The standard deviation, on the other hand, increases for all ranges of the scale of fluctuation (see Figure 92c for the 2D case; Figure 93c for the 3D case). Second, there appears to be a worst-case value of the scale of fluctuation, i.e. a critical value for which both the mean and the standard deviation of the exit gradient reach a local maximum at the same time (see Figure 92b, Figure 92d for the 2D case; Figure 93b Figure 93d for the 3D case).

Griffiths & Fenton (1998) compared the 2D and 3D results, and observed that the 3D case allows the flow greater freedom to avoid the low permeability zones, increases the averaging effect within each realisation and reduces the overall randomness of the results observed between realisations. Results were also interpreted in the context of reliability-based design, and a relationship between the traditional

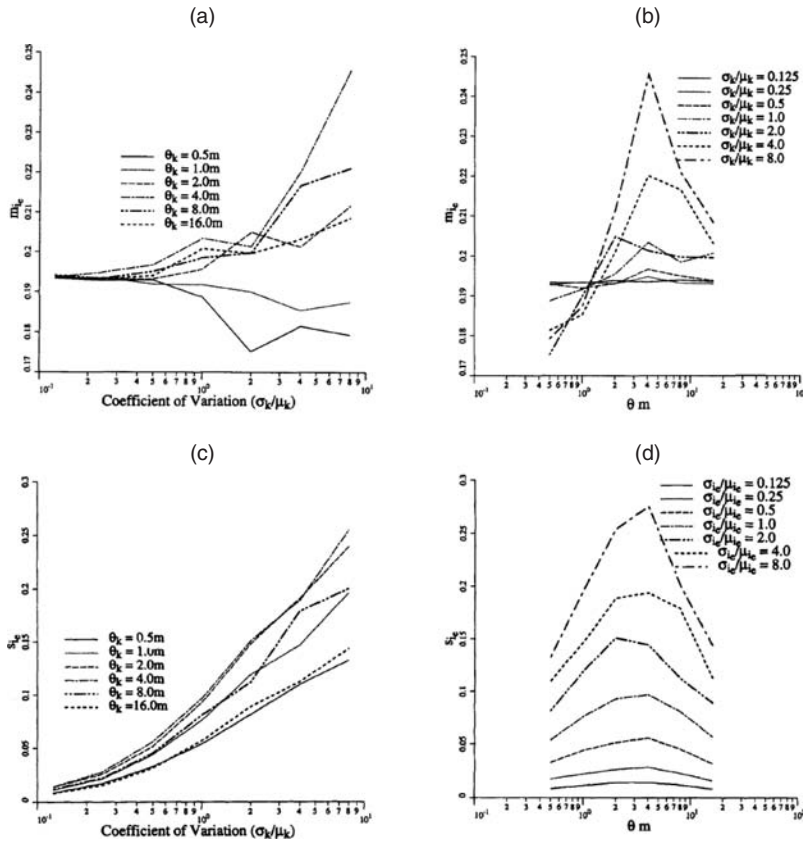


Figure 93. Results of 3D analysis of impact of variability in permeability on exit gradient from water retaining earth structures (Griffiths & Fenton 1998).

factor of safety and the probability of failure was established. Figure 94 shows the results of 1000 Monte Carlo simulations in the 2D case for the critical scale of fluctuation of 2 m and a unit coefficient of variation of the lognormal random field of permeability. It is seen that the results of simulations are very well approximated by a lognormal distribution, whose parameters may be recorded. Figure 95 shows the curves obtained by plotting the probability that the probabilistic exit gradient exceeds the deterministic exit gradient versus the coefficient of variation of the lognormal random field of permeability, both for the 2D and 3D cases and for a scale of fluctuation of 2 m. Such plot allows inference regarding the degree of conservatism in the deterministic prediction.

7.3 Stability of underground pillars

Griffiths et al. (2002) studied the influence of spatially varying strength on the failure of axially loaded rock pillars commonly used in underground construction and mining processes. Rock strength was characterised by its unconfined compressive strength using an elastic-perfectly plastic Tresca failure criterion.

An example of deformed mesh at failure is provided in Figure 96a. The spatial variability of strength (modelled as a lognormal random field with a single exponential-type correlation structure) can be appreciated, as well as the complexity in the failure mechanism resulting from the heterogeneity in strength itself. The main conclusion by Griffiths et al. (2002) was that rock strength variability can significantly reduce the compressive strength. More specifically, as the coefficient of variation of the rock strength increased, the expected rock strength decreased (see Figure 96b). Also, while the decrease in compressive strength was greatest for small scales of fluctuation, the existence of a critical scale of fluctuation, which yielded the most significant reduction in strength, was observed (see Figure 96c). Figure 96d shows that

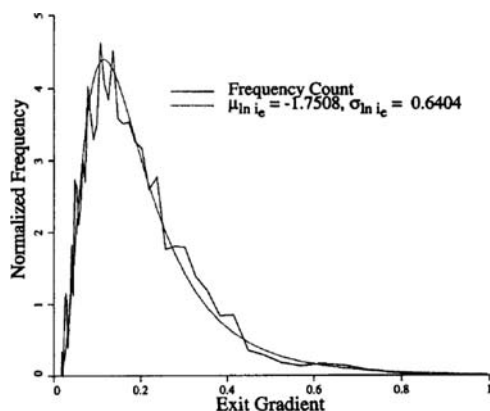


Figure 94. Histogram of exit gradients in the 2D case for a scale of fluctuation of 2 m and unit coefficient of variation (Griffiths & Fenton 1998).

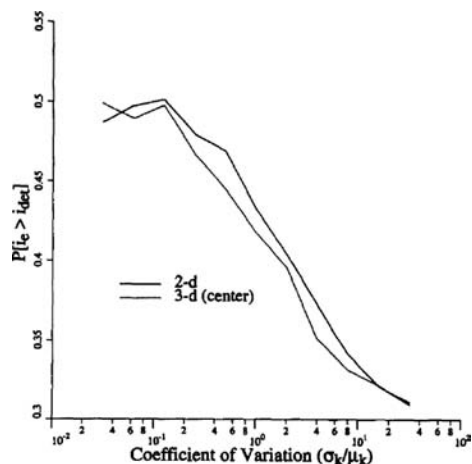


Figure 95. Probability that the probabilistic exit gradient exceeds the deterministic exit gradient (2D and 3D cases) for a scale of fluctuation of 2 m (Griffiths & Fenton 1998).

increases in coefficient of variation of strength results in an increase in the coefficient of variation of the bearing capacity factor; such positive relationship is stronger for higher values of the scale of fluctuation due to the spatial averaging effect.

7.4 Soil-structure interaction

Soil variability is a major source of soil-structure interaction problems. Breysse et al. (2005) showed the influence of spatial variability on soil-structure interaction for a number of simplified systems. For instance, they investigated the ratio of differential to absolute settlement variance for two neighbouring footings behaving independently (i.e. structural stiffness is neglected) loaded with the same load and resting on an elastic, spatially heterogeneous soil whose local stiffness is modelled using a Winkler model. The spatial variability is defined by the correlation length of the Winkler-spring stiffness (see Figure 97).

Figure 98 shows the ratio of differential to absolute settlement variance (obtained by Monte Carlo simulation) as a function of the distance between footings (L) for $B = 1.5$ m and for different correlation lengths ranging from 1.5 m to 1000 m. It may be seen that, for a given distance between footings, larger correlation distances result in statistically smaller differential settlements.

Figure 99 shows the same ratio of differential to absolute settlement variance, but as a function of the ratio of correlation length to footing size, and for various ratios of distance to footing size. For any value of the ratio of distance to footing size, the curve is bell-shaped, i.e. there is a range of correlation lengths which corresponds to statistically worst consequences.

Breysse et al. (2005) also investigated the effects of structural stiffness in soil-structure interaction accounting for spatial heterogeneity. A fully coupled soil-structure analysis was performed to investigate the behaviour of a pile groups linked by a more or less rigid slab. In the model, illustrated in Figure 100, each pile is modelled by a spring whose elastic stiffness is assumed to vary randomly. The slab has a finite flexural stiffness, and is loaded with a uniform load.

Figure 101 shows the effect of the slab depth/stiffness ratio on the load on the central pile for a constant correlation distance of pile stiffness of 10 m. The 5%, 50% and 95% fractile curves are obtained by Monte Carlo simulation. Figure 102 illustrates the effect of varying correlation distance, all other parameters constant. As in the previous case, there is a range of correlation distances (around 2 m) which result in a greater variability and, also, in a higher pile load.

In both cases, therefore, the existence of a critical correlation distance, i.e. one yielding statistically less favourable conditions, was verified. While a detailed discussion on the physical significance of the critical values for the models specifically investigated is provided in the source reference, it is sufficient

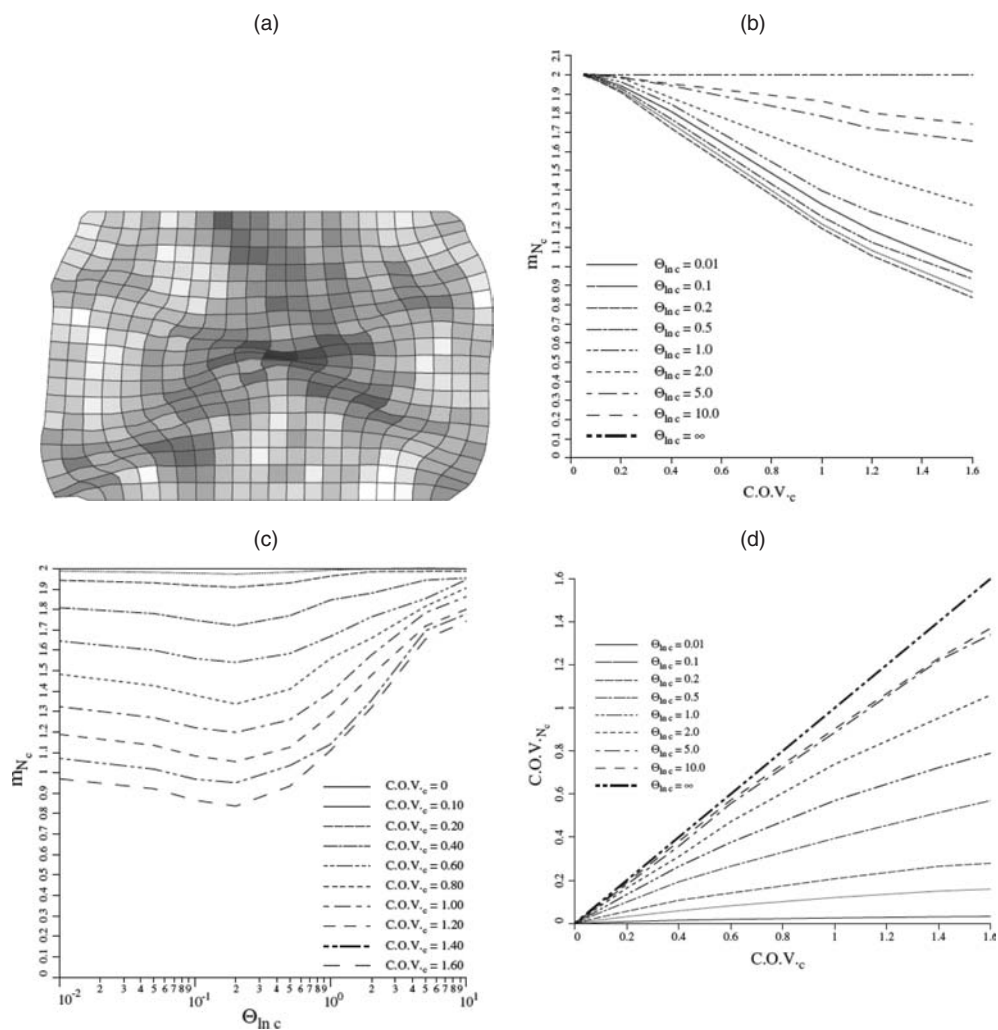


Figure 96. Selected results from the underground pillar reliability analysis (Griffiths et al. 2002).

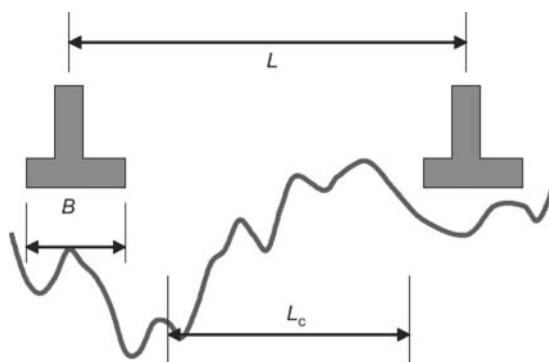


Figure 97. Reference model for the analysis of the effects of spatial variability on the settlements of two independent footings (Breysse et al. 2005).

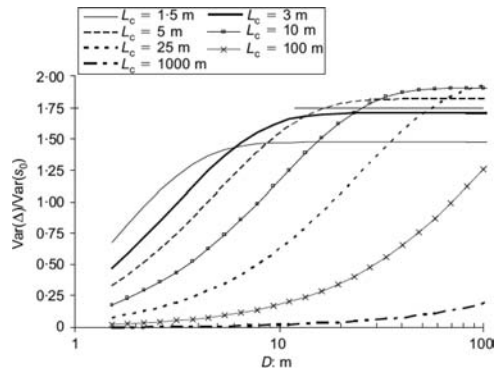


Figure 98. Ratio of differential to absolute settlement variance as a function of the distance between footings D and correlation length L_c . (Breyse et al. 2005).

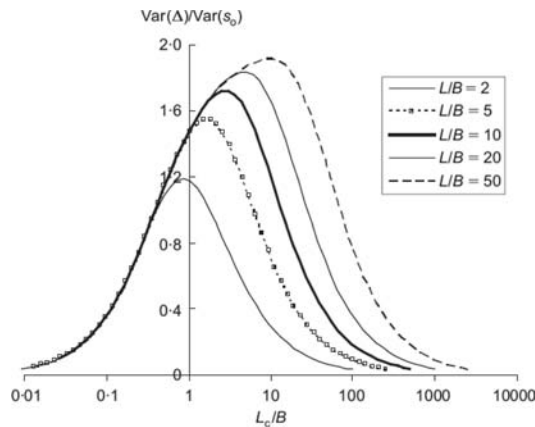


Figure 99. Ratio of differential to absolute settlement variance as a function of ratio of correlation length to footing size, L_c/B , and ratio of distance to footing size, L/B (Breyse et al. 2005).

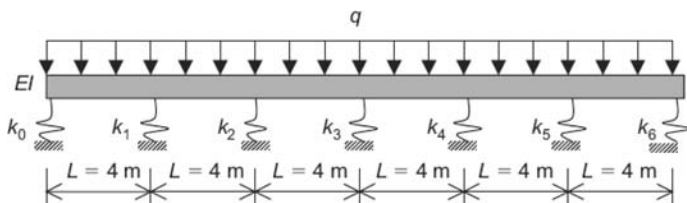


Figure 100. Model of the pile-cap system (Breyse et al. 2005).

here to acknowledge the importance of accounting for spatial variability in soil-structure interaction analyses.

Popescu et al. (2005a) investigated the differential settlements and bearing capacity of a rigid strip foundation on an overconsolidated clay layer. The undrained strength of the clay was modelled as a non-normal random field. The deformation modulus was assumed to be perfectly correlated to undrained shear strength. Ranges for the probabilistic descriptors of the random field (coefficient of variation, horizontal and vertical correlation distances, distribution functions) were assumed from the literature (a symmetric beta distribution and a gamma distribution were used comparatively). Overall settlements (including uniform and differential settlements) were computed using non-linear finite elements in a Monte Carlo simulation framework. The reference finite element mesh used in the study is shown

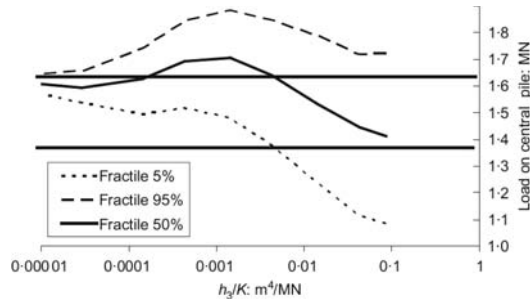


Figure 101. Effect of soil variability and stiffness ratio h_3/K on pile load (Breyse et al. 2005).

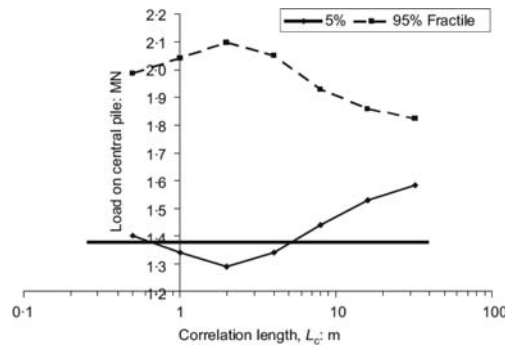


Figure 102. Effect of correlation length L_c and soil variability on pile load for central pile (Breyse et al. 2005).

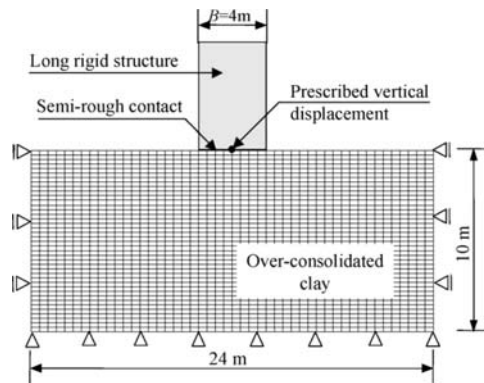


Figure 103. Reference finite element mesh and boundary conditions for the rigid strip foundation model (Popescu et al. 2005a).

in Figure 103. Anisotropy in spatial correlation is addressed, with the horizontal scale of fluctuation exceeding the vertical scale of fluctuation by one order of magnitude.

Figure 104a shows the contours of maximum shear strain for a uniform soil deposit with undrained strength of 100 kPa and for a prescribed normalised vertical displacement at centre of foundation $\delta/B = 0.1$. In this case the failure mechanism is symmetric and well-defined. The results of the analyses indicated that different sample realisations of soil properties corresponded to fundamentally different failure surfaces. Figure 104b shows an example of a sample realisation in which the spatial distribution of undrained strength is not symmetric with respect to the foundation. Hence, as could be expected, the configuration at failure, shown in Figure 104c, involves a rotation as well as a vertical settlement.

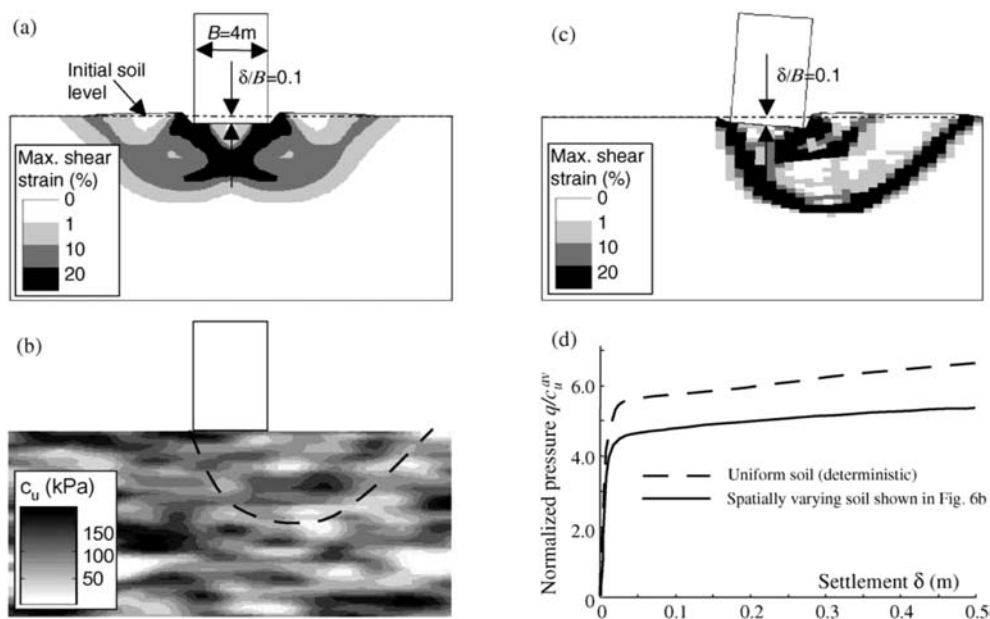


Figure 104. Selected results of investigation on homogeneous and spatially random foundation soil (Popescu et al. 2005a).

The repeated finite-element analysis allows appreciation of compound kinematisms (settlements and rotations) of the footings, which could not be inferred from deterministic bearing capacity calculations (i.e. neglecting spatial variability). In general, it was observed that failure surfaces are not mere variations around the deterministic failure surface; thus, no “average” failure mechanisms could be identified. Another relevant result was the observed significant reduction in the values of bearing capacity spatially in the heterogeneous case in comparison with the deterministic model: Figure 104d shows that the normalised pressure required to induce a given of normalised settlement is always higher in the deterministic case.

Popescu et al. (2005a) also constructed fragility curves accounting for both inherent spatial variability and epistemic uncertainty in the expected value of soil strength (i.e. induced by measurement, statistical estimation and model errors). Based on such fragility curves obtained at failure, nominal values of the bearing capacity of a spatially variable soil deposit corresponding to an exceedance probability of 5% were established for a range of values of probabilistic characteristics of both spatial variability and expected value of strength.

Fenton & Griffiths (2005) investigated the reliability of shallow foundations against serviceability limit state failure, in the form of excessive and differential settlement, both for a single footing and for two footings. Figure 105 shows cross-sections through finite element meshes of: (a) single footing; and (b) two footings. Figure 106 provides a 3-D visualisation of the finite element mesh for the two-footings case. The elastic modulus of the soil was modelled as a lognormal random field with a spatially isotropic correlation structure.

7.5 Seismic liquefaction-induced ground failure

Popescu et al. (2005b), following a number of their own past research efforts, used a Monte Carlo simulation approach involving the generation of non-normal bivariable random fields and non-linear finite element analyses to investigate the effects of soil heterogeneity on the liquefaction potential of a spatially heterogeneous soil deposit subjected to earthquake loading. Both 2D and 3D cases were addressed.

Soil heterogeneity was described using two properties: overburden stress-normalised cone tip resistance and the CPT-based soil behaviour classification index. Gamma and symmetric beta probability distributions were selected for the two parameters, respectively. A negative correlation $\rho = -0.58$ was

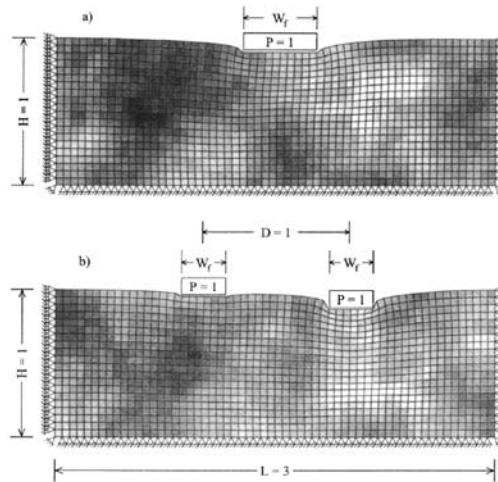


Figure 105. Cross-sections through finite element meshes of: (a) single footing; and (b) two footings founded on a spatially heterogeneous soil (Fenton & Griffiths 2005).

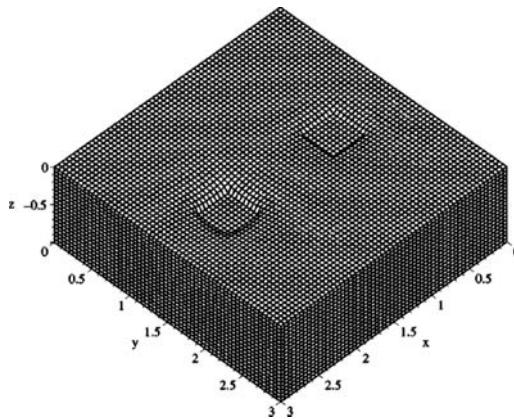


Figure 106. 3-D visualisation of the finite element mesh of spatially heterogeneous soil volume supporting two footings (Fenton & Griffiths 2005).

assumed between the two based on the results of previous studies. A separable exponential autocorrelation model was assigned to both variables. Only the saturated sand was modelled as a spatially random medium; the dry sand was modelled as deterministic. The calculations were performed for a range of seismic acceleration intensities.

The finite element meshes for the two cases are shown in Figure 107a and Figure 107b, respectively. The dimensions of the analysis domain were selected as 4–5 times larger than the vertical and horizontal correlation distances, given by the respective scales of fluctuation (2 m and 10 m, respectively). Given the small dimensions of the finite elements in comparison with the correlation distances, the variance reduction effects due to spatial averaging over the elements were neglected.

Finite element analyses were performed in two phases. In a first phase, initial effective stresses were computed by applying gravity loads and allowing full consolidation. Subsequently, nodal displacements, velocities and accelerations are zeroed and seismic input was applied at the base of the mesh. Base input accelerations were generated using a procedure by Deodatis (1996) for non-stationary random processes capable of simulating seismic accelerograms that are compatible with prescribed response spectra and having a prescribed modulating function for amplitude variation. A different input acceleration time

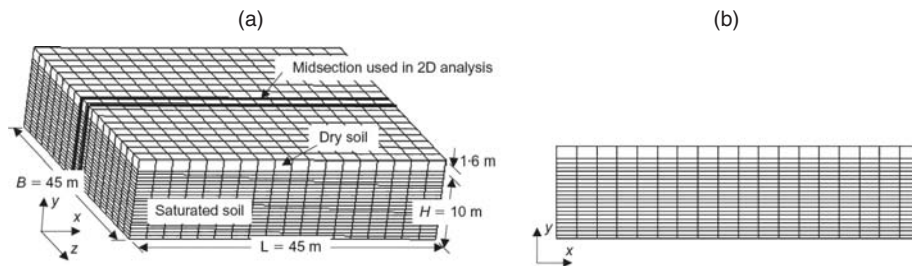


Figure 107. Finite element meshes for: (a) 3D analysis; and (b) 2D analysis (modified after Popescu et al. 2005).

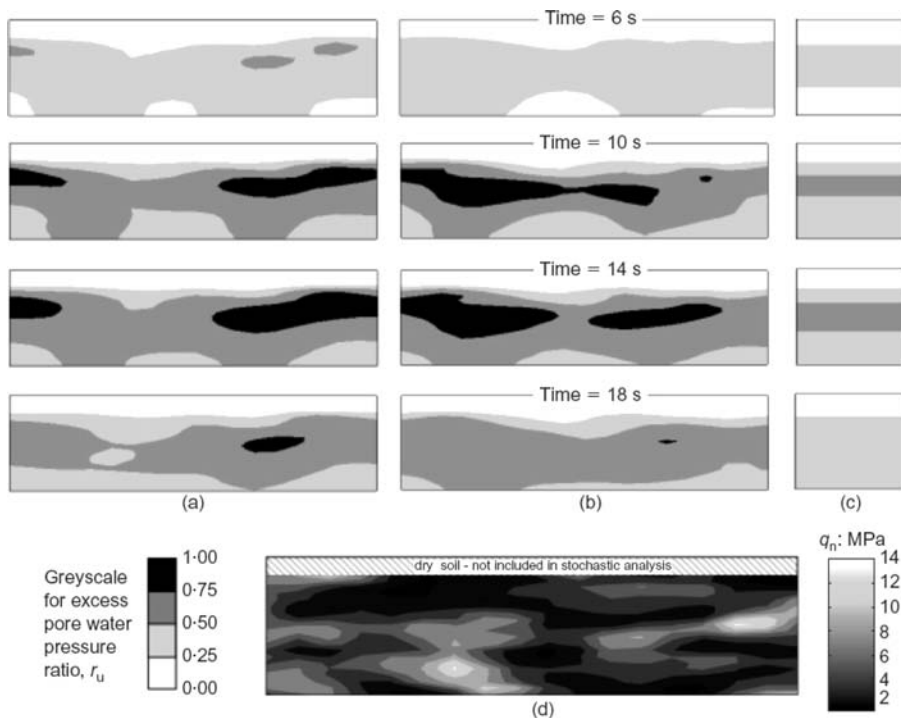


Figure 108. Comparison between computed contours of excess pore water pressure ratio with respect to the initial vertical stress for one realisation used in Monte Carlo simulation (Popescu et al. 2005b).

history is generated for each simulation. Each accelerogram was scaled according to its Arias intensity, which is a measure of the total energy delivered per unit mass (Arias 1970).

Figure 108 (Popescu et al. 2005b) provides a comparison between the computed contours of excess pore water pressure ratio with respect to the initial vertical stress for one realisation used in Monte Carlo simulation for: (a) 3D analysis (results are shown at the midsection shown in Figure 107); (b) 2D plane strain analysis; (c) deterministic model. Figure 108d shows the contours of normalised cone tip resistance corresponding to the midsection of the 3D analysis domain and to the 2D analysis domain for the same realisation.

Figure 109 illustrates the contours of excess pore water pressure ratio on the maximum-effect plane for the realisation shown in Figure 108 for 8, 12 and 16 seconds of seismic loading. Figure 109d shows the contours of normalised cone tip resistance on the maximum-effect plane for the same realisation. The white circle in (d) indicates the initial pore pressure build-up area.

Figure 110 (Popescu et al. 2005b) provides comparative diagrams of smoothed results from deterministic, 2D and 3D analyses. The plots are based on 300 Monte Carlo simulations (for both 2D and

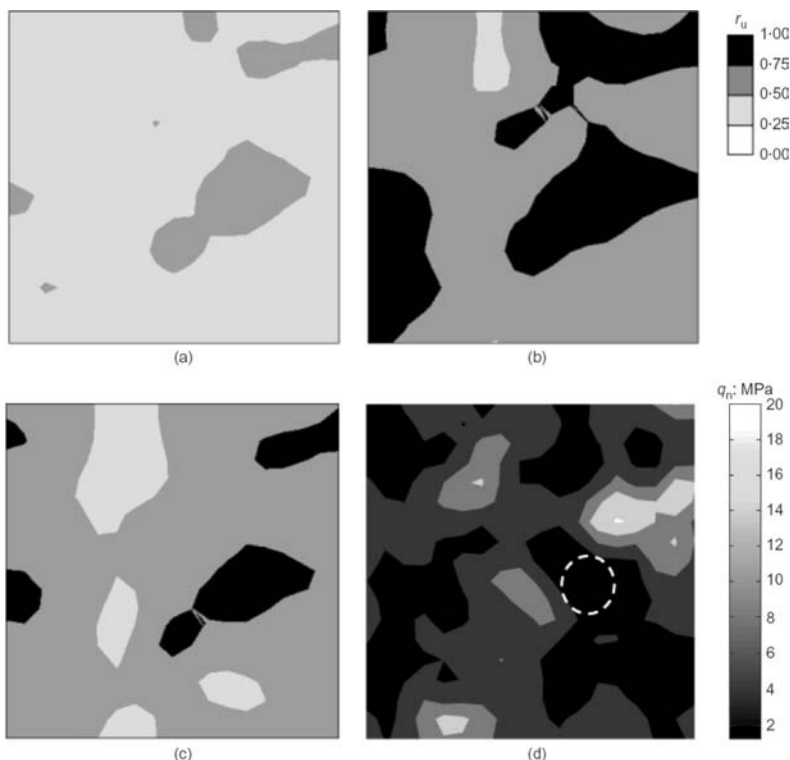


Figure 109. Contours of excess pore water pressure ratio on the maximum-effect plane for the realisation shown in Figure 108 at time: (a) 8 seconds; (b) 12 seconds; and (c) 16 seconds of the acceleration time history. Plot (d) shows the contours of normalised cone tip resistance on the maximum-effect plane for the same realisation (Popescu et al. 2005b).

3D) and corresponding deterministic analyses. The results in Figure 110 are smoothed out using a moving average window technique. Popescu et al. (2005b) noted the substantial agreement between the results of 2D (Figure 110a) and 3D (Figure 110b) analyses for excess pore water pressure ratios (both volumetric and maximum-effect plane average values). For such parameters, the deterministic analyses yielded unconservative results for low-intensity inputs and slightly greater values for higher-intensity loading. No significant differences were observed in terms of horizontal displacements (Figure 110c) while it may be seen in Figure 110d that spatial heterogeneity and 3D effects are both relevant factors in modelling the maximum liquefaction-induced ground settlement. Results suggest that a 2D model may be unconservative, and even more so the deterministic model in which spatial variability is neglected. Assimaki (2006) reported that spatial variation is important in seismic response analyses when the correlation distance is comparable to the wavelength of the incident motion and high frequency predictions are affected by complex scattering effects.

The results of the analyses were also presented in the form of fragility curves expressing the probability of exceeding a certain response threshold as a function of earthquake intensity. Figure 111 compares the results of stochastic (2D and 3D) and deterministic analysis in terms of fragility curves. The qualitative differences described for Figure 110 may be identified in Figure 111 also.

7.6 Sliding of retaining walls

Fenton et al. (2005) investigated the failure behaviour and the reliability of a two-dimensional frictionless wall retaining a cohesionless drained backfill. Soil friction angle and unit weight are modelled as spatially variable properties using lognormal random fields with single exponential-type correlation structures.

Figure 112 shows the active earth displacements for two realisations of the finite element mesh. The location and shape of the failure surface is strongly related to the presence of weaker soil zones (shown

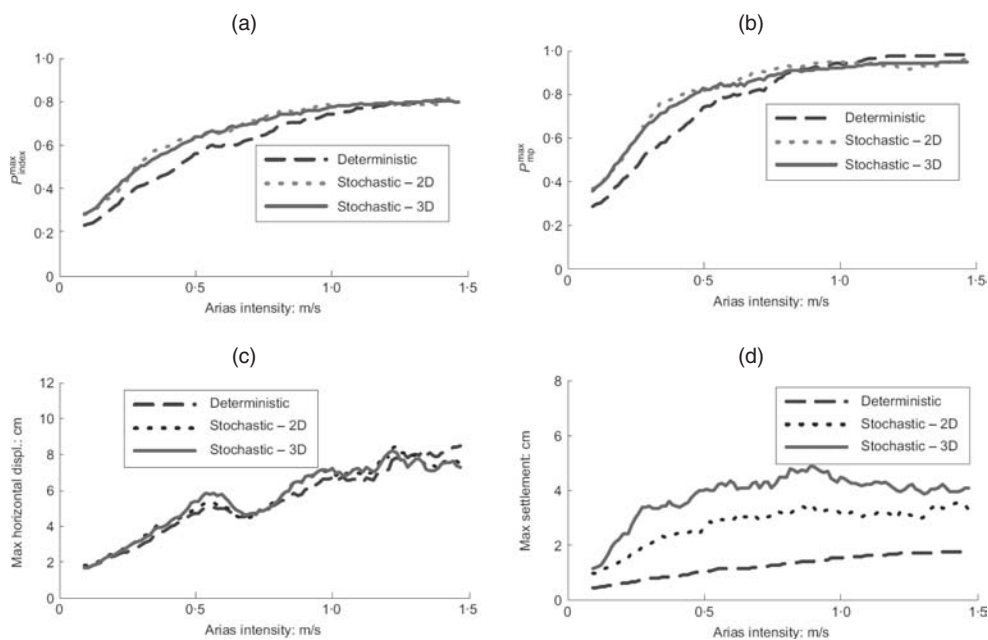


Figure 110. Comparative diagrams of smoothed results of deterministic, 2D and 3D analyses: (a) maximum value of the volumetric average of excess pore pressure ratio; (b) maximum value of excess pore pressure ratio on maximum-effect plane; (c) maximum predicted horizontal displacement at ground level; and (d) maximum predicted settlement at ground level (Popescu et al. 2005).

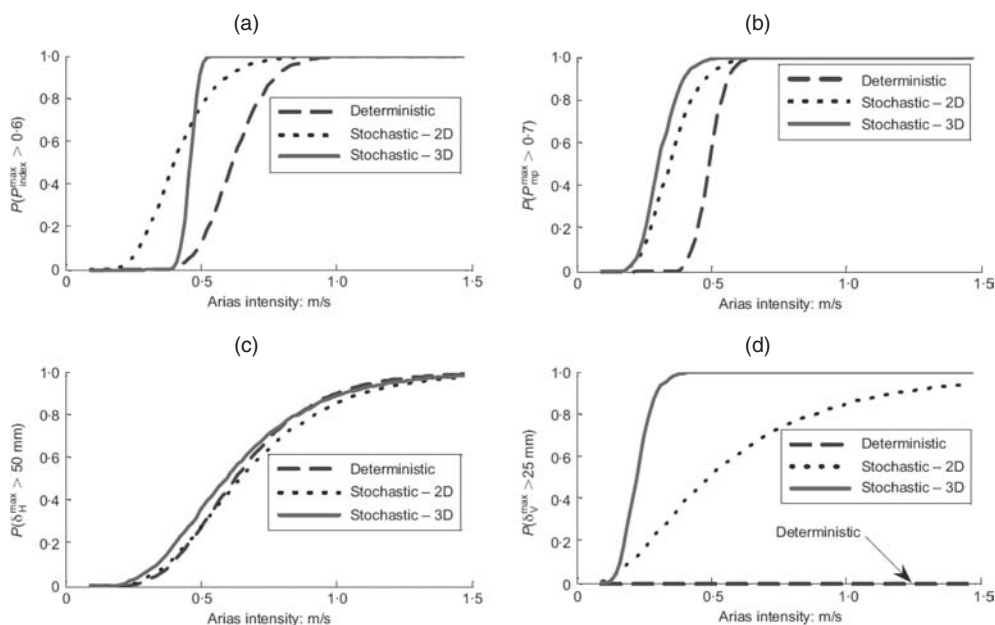


Figure 111. Stochastic (2D and 3D) and deterministic analysis in terms of fragility curves: (a) probability that maximum value of volumetric average of excess pore pressure ratio exceeds 0.6; (b) probability that maximum of maximum-effect plane average of excess pore pressure ratio exceeds 0.7; (c) probability that maximum horizontal displacement exceeds 50 mm; (d) probability that maximum settlement exceeds 25 mm (Popescu et al. 2005b).

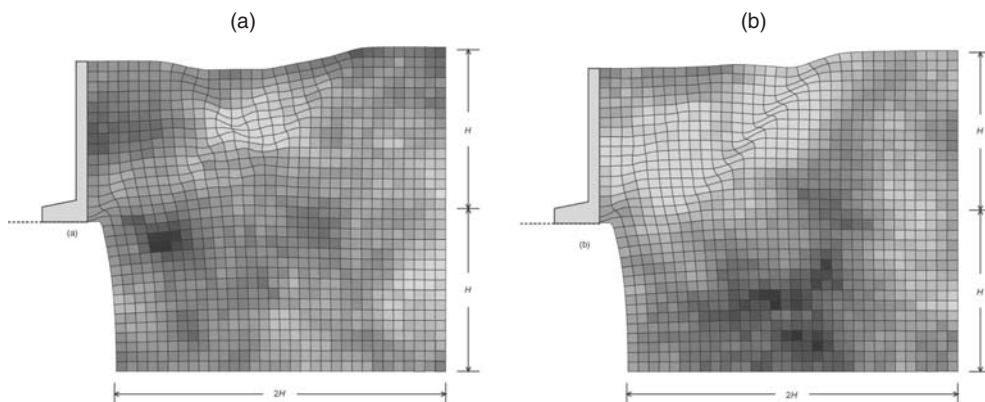


Figure 112. Active earth displacements for two realisations (both having the same correlation distance and coefficient of variation) of the random field of soil friction angle (Fenton et al. 2005).

in lighter colours in the finite-element mesh) and is, in both cases, markedly different from the simple shapes which are assumed in earth pressure theory (e.g. planar in the Rankine model).

In the study by Fenton et al. (2005), unit weight and friction angle random were first assumed to be uncorrelated, then strongly correlated. An analysis of the results indicated that the correlated case yielded lower probabilities of failure than the uncorrelated case, which can thus be regarded as conservative. Again, a critical value of the scale of fluctuation, for which active loading on the retaining wall was maximised, was observed.

8 CLOSING STATEMENTS

Newer design codes recognise and address the uncertainties in soil properties and engineering models. Soil variability then assumes an increasingly important role in practice and research. This should be seen as a positive evolution of the geotechnical discipline, as addressing uncertainty in a consistent manner allows more economic and rational design.

The paper provides an overview of selected techniques for modelling the inherent variability of soils. A perspective as practical as possible was pursued, with wide reference to available literature. Examples from probabilistic slope stability analyses were illustrated to highlight the benefits and limitations of approaches with various levels of complexity. Updated data tables were provided for illustration purposes. Most statistics available in the literature are strongly site- and case-specific, and the data should be examined with caution if they are to be applied at other sites.

Research may help simplify the use of variability-modelling techniques, thus assisting the practitioner. However, even the most powerful modelling technique can yield unreliable results if input data are insufficient in quantity and quality. Geotechnical practice makes use of data sets which invariably indicate variability in any soil property. The variability information is often lost in the characterisation and design processes. A first step towards an uncertainty-based approach in geotechnical practice could be the explicit reporting of properly obtained data statistics.

At present, research efforts focus on a variety of aspects of soil variability modelling. Advanced simulation techniques, for instance, allow bypassing some of the barriers encountered in past efforts. Enhanced capabilities of computing tools and use of sophisticated integrated methodologies make it possible to model with increasing realism the behaviour of complex geotechnical systems. Geotechnical practice, on the contrary, still largely relies on deterministic approaches. The gap between geotechnical research and practice needs to be narrowed.

It is perhaps illusory that such gap may ever be eliminated. However, a greater acceptance of variability as a major actor in geotechnical practice would help reduce such gap. Reduction of the gap should occur from both sides: research should merge the mathematical techniques and geotechnical data into a more readily usable format; practice should increasingly recognise the importance of addressing data in the light of uncertainty, and accept the necessity to acquire additional competence regarding the statistical treatment of data.

There is little doubt that a shift to an uncertainty-based perspective is taking place. The joint effort of researchers and practitioners should aim towards a full recognition of the benefits of such development.

ACKNOWLEDGEMENTS

The authors wish to acknowledge the collaboration of Jostein Jerkø (on internship at NGI from NTNU) in reviewing the literature and compiling the data tables, as well as the assistance by Tini van der Harst from NGI in formatting the paper.

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Alluvial clays

Engineering characteristics of Taipei Clay

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ABSTRACT: Taipei City is located within the Taipei Basin on the northern part of Taiwan. The cohesive deposit of the soils is referred as Taipei Clay, which is the soil deposit of most significant engineering concern. Due to rapid development of the city, major building and infrastructure projects demand a comprehensive understanding of the soil deposits. The engineering characteristics of Taipei Clay have been extensively investigated in those major projects. In this paper, the geology of Taipei City will be reviewed first. This section is followed by the results of geotechnical zoning of Taipei City. It provides an overall picture of the extent and variability of soil deposits. Then, detailed studies of Taipei Clay are presented, including state of the clay, advanced laboratory test results, and in situ test results. Finally, significant engineering problems encountered by the geotechnical engineers are discussed.

1 BACKGROUND

Taipei City is the most populated city in Taiwan. It is located in the northern part of Taiwan Island and occupies more than half of the area of the Taipei Basin Figure 1. Due to rapid economic development of Taiwan over the past several decades, many high-rise buildings and major infrastructures have been built in Taipei. All these constructions have been carried out in the recent Quarternary deposits within the Taipei Basin.

Before 1960s, construction within the Taipei Basin was carried out with minimum knowledge of the soils. Most studies on the deposits are from geologic point of view. The first systematic engineering study was conducted by Hung (1966). He named the whole Sungshan Formation, the Quaternary deposits of the Taipei Basin, as the “Taipei Silt”. He also identified six interbedded silty clay and silty sand layers for the Quarternary deposits. Results of a clay mineralogy study and a summary of soil index properties were reported in his report. Groundwater conditions have also been briefly discussed in that study.

More than a decade after Hung’s study, Moh & Ou (1979) conducted a more comprehensive study on Sungshan Formation with better sampling techniques and more sophisticated laboratory equipments. The properties of six alternating layers of silty clay and sand of Sungshan Formation were further investigated. The particular interest of the paper is the emphasis of silt content. The “six-layer model” introduced by Hung (1966) has been adopted in this paper. Moh & Ou (1979) still used “Taipei Silt” to describe the whole Sungshan Formation: *“The subsoil stratum which is of prime concern in this paper is the Taipei Silt. In general, the uniformly distributed soil stratum is composed of alternative layers of low plasticity silty clay (ML-CL or ML) and silts (ML), interstratified with fine sand layers containing high silt content. The silt content in the various layers ranges from 15% to 95%”*. Moh & Ou (1979) is the first paper, from an engineering point of view, to highlight the impact of deep well pumping and its associated ground subsidence on the construction of foundations and other underground structures. It should be noted that piezometric pressures measured at that time (1979) are much lower than the assumed hydrostatic pressure distribution due to significant deep well pumping.

In early years, most studies have been carried out around the Taipei Main Station where is the downtown area of Taipei at that time. However, with more projects in different places of the Taipei Basin, it is found that the thickness of each stratum varies and properties of the soils also vary. The “six-layer model” is still applicable, but it needs refinement to better delineate the engineering properties of soils in the Basin. The first city-wide geotechnical mapping was conducted and completed by Moh and Associates, Inc. (MAA) in mid-1980’s (MAA 1987). This study divided the city area into six geotechnical zones based on a very scrupulous review of physical properties of soil samples and the depositional environment of three rivers



Figure 1. Satellite Image of the Taipei Basin (Copyright (c) CSRSR/CNES, 2004, Courtesy CSRSR, NCU).

flowing into the Basin Figure 2. This study had great implications for further researches as well as for engineering practice. Several geotechnical zoning and microzoning works have been conducted on the basis of this study (e.g. Cheng 1987, Lee 1996). Results of this study have been used in the planning of major engineering projects in Taipei City, such as the Taipei Rapid Transit Systems (TRTS). Since it further validated the “six-layer model”, geotechnical engineers in Taipei can better describe the soil layer they refer to, say, Layer II of K1 Zone. In addition, a series of engineering correlations have been established (MAA 1987).

As the TRTS got started, there were much stronger needs to understand behaviors of the cohesive soils in the Taipei Basin. A series of studies have been carried out (Chin et al. 1989, Liu et al. 1991, Chin & Liu 1997) to evaluate whether the silty clay in Taipei possesses the normalized behavior and how significant is the anisotropy of the undrained shear strength. Test results indicate that the cohesive deposits in Taipei behave just as a low plastic clay, which implies that the cohesive deposits in Taipei can be generally modeled as other clay deposits. One of the major endeavors in the study of the soil properties was the Designated Task of TRTS. It was carried out by MAA, the Geotechnical Engineering Specialist Consultant (GES) of TRTS, with the participation of MIT and Golder Associates. Some high quality samples have been tested at MIT. These test results can be treated as a set of “benchmark” test results for further studies of Taipei soils. Later on, soil parameters of the MIT-E3 model have been derived from these results and used in an advanced numerical analysis of deep excavations.

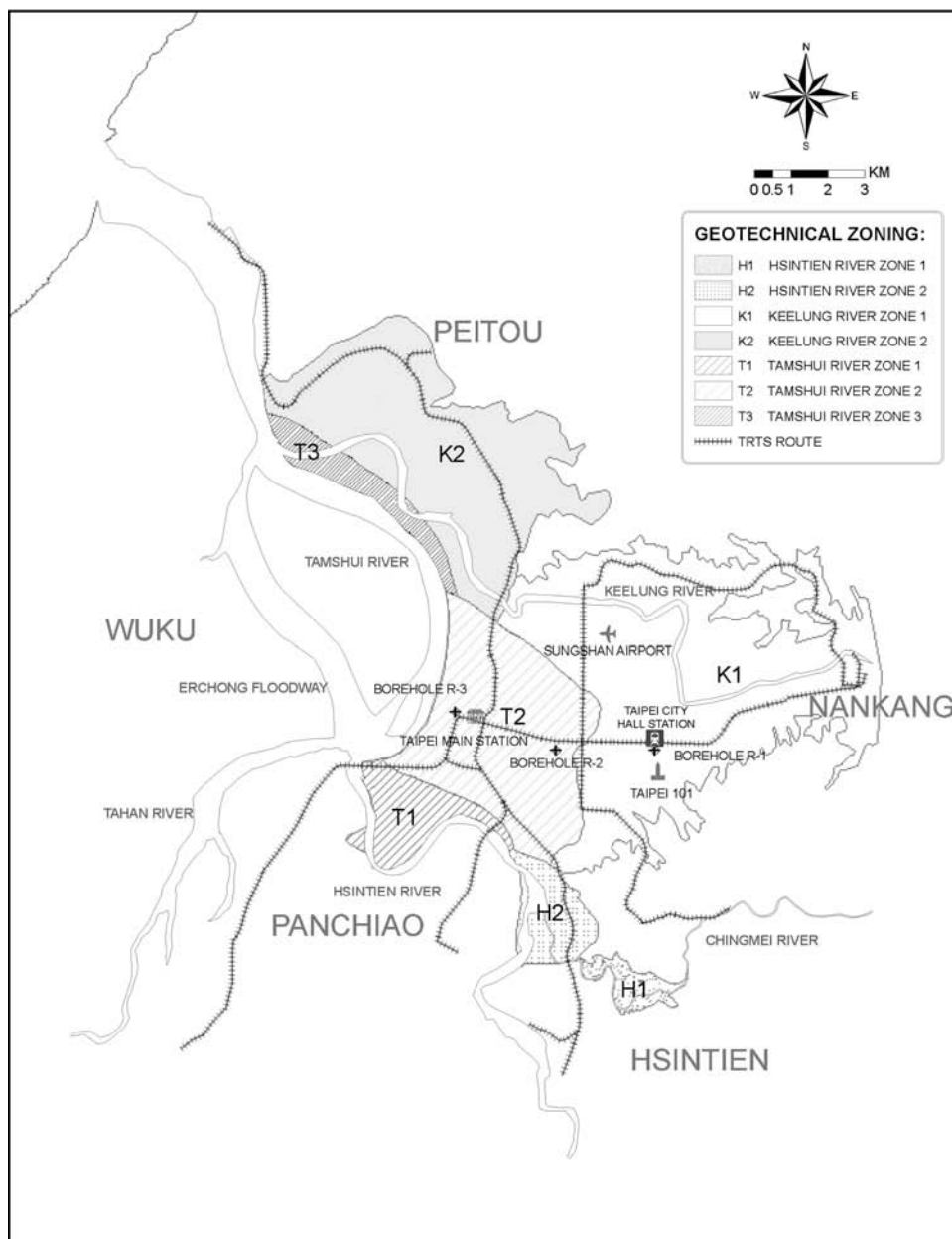


Figure 2. Geotechnical zoning of the Taipei Basin and the priority network of TRTS.

After the completion of the initial TRTS network, construction in Taipei City has continued to evolve into a new era that demands high-quality and advanced sampling, testing, and modeling technology for much more difficult construction environments.

This paper presents a comprehensive overview of behaviors and properties of Taipei Clay collected from numerous field and laboratory experiences. Lessons learned from the past and information summarized in this paper can provide better insight and guidelines for future construction in Taipei City. It has to be noted that “Taipei Clay” is the cohesive deposits of Sungshan Formation in the Taipei Basin. Furthermore, Layer IV of the Sungshan Formation often occurs at about the invert of open excavations and is the medium through which tunnels will pass for the TRTS. Hence, Layer IV clay has been the

focus of past studies, which rendered significant data for this paper. For Layer IV materials in K1 Zone, liquid limit (w_L) is about 35% and plasticity index (I_p) is about 13% (MAA 1987). It is interesting to note that these index properties are not much different from those of Boston Blue Clay. For Boston Blue Clay, the liquid limit is about 42% and the plasticity index is about 19%. Hence, when possible, relevant engineering properties of these two clays will be compared.

In this paper, the geology of Taipei City will be reviewed first. This section is followed by the results of geotechnical zoning and information on stratigraphy of the city. It provides an overall picture of the extent and variability of soil deposits in the Taipei Basin. Statistical characterization of physical properties for soils in each zone concluded from the mapping study also will be given. Then, detailed studies of Taipei Clay are presented, including state of the clay, advanced laboratory test results, and in situ test results. Finally, significant engineering problems encountered by the geotechnical engineers are discussed.

2 GEOLOGY OF TAIPEI CITY AREA

2.1 *Geology of the Taipei Basin*

2.1.1 *Overview*

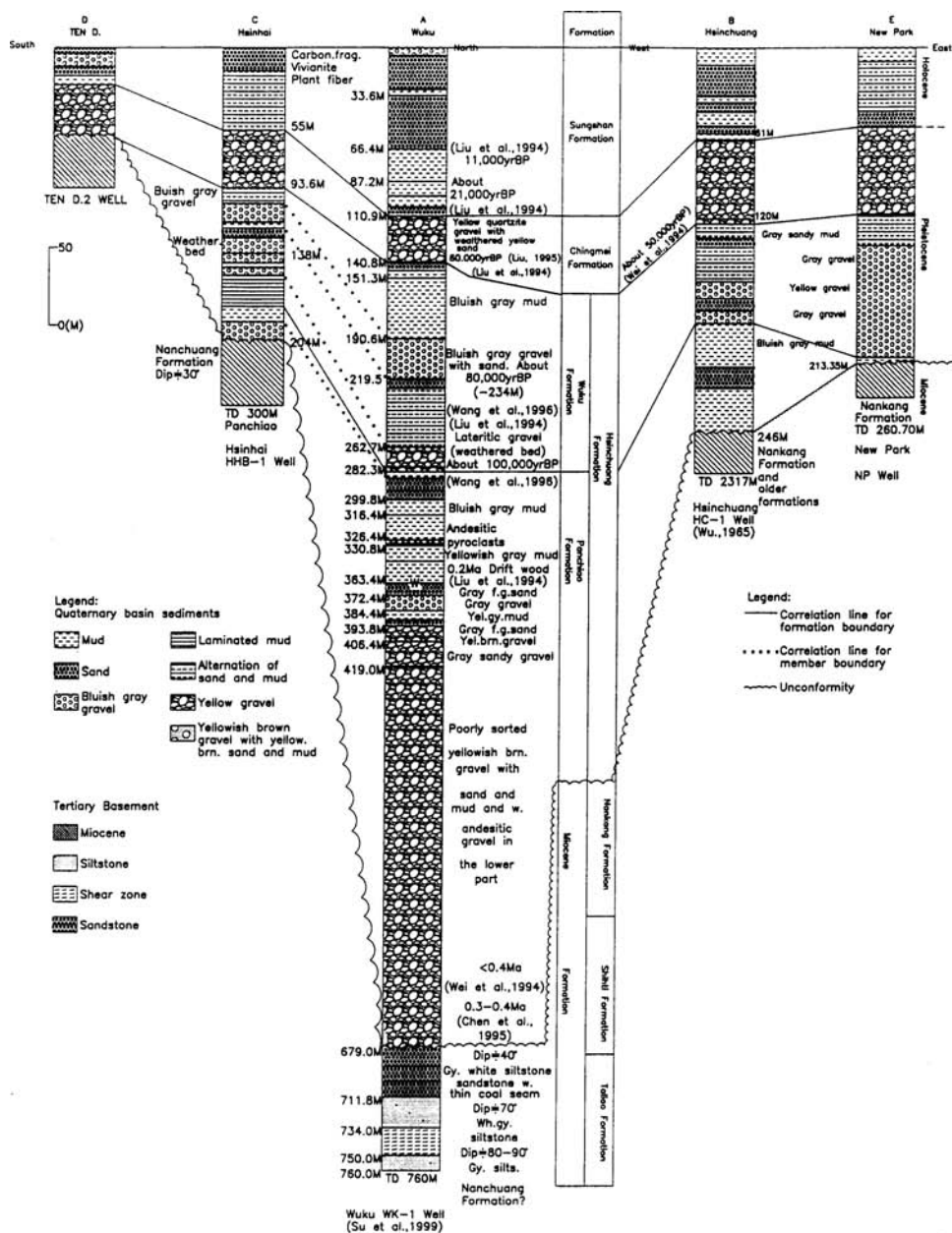
The Taipei Basin was formed as a result of reverse and normal faulting in northern Taiwan. The reverse faults, including the Hsinchuang Fault on the northwest, the Kanchiao Fault on the northeast, and the Taipei and Hsintien Faults on the southeast of the Taipei Basin, are generally trending NE or ENE with thrust upward from southeast to northwest. The most significant normal fault, Shanchiao Fault on the west of the Taipei Basin, is trending NWN with a dip toward ENE. The geology surrounding the basin can be distinguished into three geological zones Figure 3, the Tatun Volcanic Group to the north, the Tertiary Sedimentary rocks to the southeast, and the Quaternary Linkou Tableland to the west (Wang Lee et al. 1978, Teng et al. 1994).

The basin is filled with “unconsolidated” Quaternary and Recent sediments Figure 4. Most materials deposited in the basin have been derived mainly from the weathering products of the peripheral Tertiary formations through the three major rivers, the Tahan River, the Hsintien River, and the Keelung River. They were also partly from the eruption and weathering products of the Pliocene to Pleistocene andesite and basalt of the Tatun Volcano Group and the Kuanyinshan Volcano. Taipei Clay is referred to the cohesive deposits of the Quaternary sediments in this paper.

2.1.2 *Depositional environment*

The depositional process and environments of the Quaternary sediments in the Taipei Basin (Table 1) are summarized in ascending order as follows (Chou 2005):

- (a) Panchiao Formation: The basal gravel beds, sandy and muddy sediments of the Pleistocene Panchiao Formation in the Taipei Basin were deposited under the alluvial fan, fluvial and lacustrine environments. The thickness of the Panchiao Formation ranges from 0 to 397 m and varies with basal topography of the Miocene basement in the Taipei Basin. The Panchiao Formation is the thickest in the Wuku area of the western part of the Taipei Basin, and it gradually thins out toward the northeast and the southeast in the Basin.
- (b) Wuku Formation: The gravelly, sandy and muddy sediments of the late Pleistocene Wuku Formation is also the thickest in the Wuku area, being around 142 m in thickness. It gradually thins out toward the northeast and the southeast in the Basin.
- (c) Chingmei Formation: The yellow quartzite gravel bed of the late Pleistocene Chingmei Formation, in the Taipei Basin, were deposited entirely under the alluvial fan environment. The thickness of the Chingmei Formation ranges from 30 to 138 m. The greatest thickness of Chingmei Formation is in the Chingmei area, and it gradually thins out toward the north.
- (d) Sungshan Formation: The sandy and muddy sediments of the late Pleistocene to Holocene Sungshan Formation in the Taipei Basin have been deposited under the fluvial, lacustrine, estuary and brackish water bay environments. The thickness of the Sungshan Formation ranges from about 20 to 120 m. The greatest thickness of Sungshan Formation is in the Wuku area, and it gradually thins toward the southeast and the northeast. Near the end of the deposition of the formation, the brackish water gradually retreated from the basin. Numerous plants grew in the marsh and were later buried to form the peat beds.



Note: well locations shown in Figure 3

Figure 4. Stratigraphic column of the Miocene and Quaternary in the Taipei Basin (formation data from Wang Lee et al. 1978, Teng et al. 1994, Wu 1965, Liu et al. 1994, Wei et al. 1994, Chen et al. 1995, Wang et al. 1996, Su et al. 1999).

2.1.3 Rate of sedimentation

According to the radiocarbon dating results (Liu et al. 1994), rates of the sedimentation of the late Pleistocene to Holocene in Sungshan Formation are estimated to be from 1.10 to 6.03 mm/year, and the greatest rate happened in the Wuku area to the west of Taipei.

The sedimentation rates of the late Pleistocene Chingmei Formation are estimated to be from 0.80 to 1.60 mm/year from results of radiocarbon dating made by Liu et al. (1994). The rate is slightly higher

Table 1. Depositional environment and sedimentation rate of Quaternary sediments in Taipei Basin (Chou 2006).

Formation	Depositional environment	Sedimentation rate (mm/year)
Hsinchung		
Panchiao	Alluvial fan-fluvial-lacustrine	0.02–1.98
Wuku	Alluvial fan-fluvial-lacustrine	1.58–3.54
Chingmei	Alluvial fan	0.80–1.60
Sungshan	Fluvial-estuary-lacustrine-brackish water bay	1.10–6.03

in the Chingmei area of southern Taipei Basin. The sedimentation rates of the late Pleistocene Wuku Formation are estimated to be from 1.58 to 3.54 mm/year based on results of radiocarbon dating made by the researchers (Liu et al. 1994, Wang et al. 1996). It is slightly higher in the Wuku area of western Taipei Basin.

The sedimentation rates of the late Pleistocene Panchiao Formation are estimated to be from 0.02 to 1.98 mm/year based on the results of radiocarbon and $^{40}\text{Ar}/^{39}\text{Ar}$ dating results (Wei et al. 1994, Wang et al. 1996). The greatest rate is in the Wuku area and gradually reduces from there toward the east and the south of the Taipei Basin.

The sedimentation rates of Quaternary sediments in the Taipei Basin are also summarized in Table 1. Generally speaking, the sedimentation rate of the Quaternary sediments in the Taipei Basin is slower since 0.4 Ma and gradually increases toward the Holocene.

2.1.4 *Water depth*

The deposition of the late Pleistocene to Holocene Sungshan Formation in the Basin was under the fluvial, estuary, lacustrine and brackish water bay environments characterized by shallow depth, weak current and quiet water. These conditions are verified by the texture and fossil content of the sediments. The Recent water level of the Tamshui River and the Keelung River ranges from 0.1 m to 1.3 m during the high tide and from –0.4 m to 0.06 m during the low tide.

2.1.5 *Stress history*

Since the Taipei region subsided around 0.3–0.4 Ma and the Taipei Basin formed, the area has undergone a continuous sedimentation process. Hence a new land, the Taipei Basin, has been developed under the alluvial, fluvial, lacustrine and brackish water sedimentation and probably without recognizable erosion.

2.1.6 *Tectonics*

Due to the oblique collision between the Eurasian Plate and Philippine Sea Plate since the Pliocene to Pleistocene, the Tertiary sequences have been folded and faulted, and the northeast trending Chinshan-Hsinchuang Fault, the Kanchiao Fault, the Taipei Fault, the Hsintien Fault and the Chengtzuliao Fault were formed in Taipei area successively (Figure 3). And then the Shanchiao Fault, the Neihu Fault, the Tachih fault, the Kungkuan Fault and the Taita Fault were formed due to the great subsidence and normal faulting of the Taipei area since around 0.4 Ma (Chou 2005).

The eruptions of the Tatun and Kuanyinshan Volcanoes caused the great subsidence of the Taipei region. The Shanchiao Normal Fault has been successively formed parallel with the Hsinchuang Fault along the northwestern border of the Taipei region since around 0.3–0.4 Ma, and the Taipei Basin has been formed. The Shanchiao Fault is still active based on the seismic exploration and the observations on the subsidence of the Taipei Basin (Wang et al. 1994, Wang et al. 1995, Wang et al. 1996, Yu et al. 1994, Chou 2005).

Due to gradual compaction of the thick and loose late Quaternary sediments as a result of sediments loadings and shakings by the successive earthquakes, the southeastern hanging wall of the Shanchiao Fault gradually subsided to form the Wuku gravity normal growth fault. These faults offset the late Quaternary loose formations for about tens of meters nearly vertically, and they are still moving and active (Wang et al. 1996, Wang et al. 1999, Yu et al. 1994, Chou 2005).

2.1.7 *Temperature*

The pollen analysis of the Wuku no.13 core in the western Taipei Basin provided a detailed record of the paleoclimatic changes in the Taipei Basin (Tseng & Liew 1999). The forest-grassland pollen assemblage

demonstrated that the cold-dry period of the last glacial changed at about 17100 yr. BP (79.8 m in depth) when arboreal pollen elements increased. The shallow water environment appeared in the Basin because of the rise of the aquatics since 12400 yr. BP (69.2 m in depth). The climate trended into warm-wet at about 10800 yr. BP (65 m in depth) due to dominance of Fagaceae. An episodic cool phase in about 10200 yr. BP (63.4 m in depth) was probably correlated to Younger Dryas cooling event when the cool-temperature elements increased, then yellow erosive soil appeared and water level lowered. Obviously, increased tropic-subtropical and warm-wet arboreal pollen elements, the coarser sediments, and the highest sedimentation rate indicated the warm-wet climate since 9100 yr. BP (50.4 m in depth). The abnormal at about 7300 yr. BP (39.8–23.4 m in depth) may due to the intensification of monsoon in the warm-wet climate. There were created clearances prior to the changes of vegetation probably due to the human activity at about 3800 and 3500 yr. BP (10.65 and 10.1 m in depth). According to the pollen assemblage of the palynological study by Shaw et al. 1999, the upper part of Sungshan Formation is considered to be the most hot and humid stage in the Taipei Basin.

2.1.8 Bioturbation

The foraminifer, ostracod and mollusk fossils are found in almost every member or lentil of Sungshan Formation in the Taipei Basin. The foraminiferal fauna in Sungshan Formation is composed largely of benthonic forms with subordinate pelagic species. The Taipei Basin foraminiferal fauna is characterized by the predominance of benthonic species, especially two genera, *Streblus* and *Elphidium*. The abundant benthonic components and the extremely scarce pelagic species imply that the depositional environment of the sediments is mainly shallow water. Two leading species, *Streblus annectens* and *Elphidium tainanensis* are characteristics of the brackish water form (Huang 1962).

The mollusks, *Cotbicula formosana*, *Ostrea gigas*, *Placuna placenta*, and *Sinonovacula constricta* were found in Sungshan Formation. *Cotbicula formosana* was found by Tan (1939) in the present estuary of the Tamshui River (Kuroda 1941).

The other fossils found in Sungshan Formation consist of Balanids, Ostracods, fish bone, and planet remains (Huang 1962). Therefore, Sungshan Formation is weakly bioturbated based on the containing fossils.

2.1.9 Local geological features: engineer's concerns

In the Taipei Basin, methane and drift woods are the two unique geological features (Moh & Hwang 1997). Methane was encountered in several boreholes during investigation. Its presence is related to the capture of gas in dooms encapped by impermeable clays (Lin et al. 1997). In one case, during the drilling of a borehole of the Chungho Line of TRTS, the eruption sent a jet of methane-water mix to a maximum height of about 10 m into the air and continued for more than 3 days. Contractors have been warned for the possibility of encountering methane in tunnel drives and possible consequences. As a precaution, specifications require the concentration of methane be continuously monitored during tunneling of TRTS and shield machines be equipped with detective devices and alarm systems. Gas problem has also been encountered during some basement excavations in Taipei (Liu 2006). It should be noted that methane could be encountered at locations all over the Taipei Basin.

Drift woods were recovered in numerous excavations and were responsible for a few accidents encountered during tunneling (Lin et al. 1997, Ju et al. 1997). Chunks of 1 m or so in diameter were frequently encountered and they may be as long as 5 m. A piece of wood found at a depth of 9 m in Observation Well 2 (W-1797) in Panchiao dated back to 6760 years and that found at a depth of 23 m dated back to 7,950 years (Liew 1994). The presence of drift woods was well recognized and has been well reported, however, it is still very difficult to predict the exact locations and depths of drift woods beforehand. The drift wood problems could be very serious for shield tunnelings. All the shield machines adopted for TRTS have sufficient strength and power to cut through the woods as long as they are not too large in size. However, there was one occasion in which the presence of drift wood caused the ground to collapse as the movement of the shield was hampered and soft clay kept on moving into the earth chamber. Finally, a hole of 5 m in diameter occurred right above the head of the shield machine. Workers entered the earth chamber and recovered two pieces of drift wood which were 500 mm and 400 mm in length, respectively (Moh & Hwang 1997).

2.2 Composition

2.2.1 Form of clay fraction

The clay fractions of Sungshan Formation in the Taipei Basin are mainly composed of non-clay minerals, clay minerals, and a small amount of biogenic debris. Most of the non-clay minerals are concentrated largely in the silt-size and sand-size fractions. The non-clay minerals are composed mainly of quartz, coal,

and rock fragments with a small amount of feldspar, muscovite, biotite, limonite, vivianite, hornblende, pyroxene, and undetermined minerals (Huang 1962, Chou 2005). The minerals of the clay-size fraction finer than 2 microns of Sungshan Formation were studied by X-ray diffraction. The clay minerals are mainly composed of illite with some chlorite. The rock fragments are mainly of fragments of fine sandstone, quartzose sandstone, slate, shale, and schist (Huang 1962).

The biogenic debris in Sungshan Formation is composed of foraminifers, ostracods, mollusks, bal-anids, coal, peats, pollens, and spores. The foraminiferal fauna in Sungshan Formation is composed largely of benthonic forms with subordinate pelagic species. The Taipei Basin foraminiferal fauna is characterized by the predominance of benthonic species, especially two leading species, *Streblus annectens* and *Elphidium taiwanensis* (Huang 1962). The ostracods are mainly of *Cytheris* sp. In the present assemblage of the mollusks, *Corbicula formosana*, *Dentalium* cf., *robustum*, *Dosinia gruneri*, *Natica maculosa*, *Nassarius hepaticus*, *Ostrea gigas*, *Ostra* spp., *Placuna placenta*, *Pyrene bella*, and *Sinonovacula constricta* are found in Sungshan Formation. The fossil *Balanus* aff. *amphirata* was found in association with mollusk fossils.

2.2.2 Organic content and form

The peat beds were found in the upper part of Sungshan Formation in the southern Taipei Basin. The coal fragments derived probably from the Miocene coal-bearing beds were found in Sungshan Formation.

2.2.3 Mineralogy

The clay-sized fraction of the late Quaternary sediments in the Taipei Basin is mainly composed of clay minerals with minor amount of non-clay minerals, which are usually quartz, feldspars, limonite (mainly goethite), gibbsite, and calcite etc. The clay minerals found in the sediments are illite, chlorite, kaolinite, montmorillonite, bermiculite, and mixed-layer clay minerals. As a whole, the illite, chlorite, and mixed-layer clay minerals are three major clay minerals in the sediments. On the contrary, kaolinite and montmorillonite are the minor components. Illite and chlorite have an obviously increasing trend of abundance, from the Lower Member of Hsinchuang Formation (Panchaio Formation) up to Sungshan Formation, while kaolinite has an obviously decreasing trend. Montmorillonite and mixed-layer clay minerals roughly have a decreasing tendency upwards. This phenomenon is closely related to the gradual change from the mainly alluvial depositional environment at the earlier stage to the brackish lacustrine environment at the later stage of the sedimentation of the Taipei Basin (Lin & Chen 1999).

During the subsequent sedimentation through the Upper Member of Hsinchuang Formation (Wuku Formation), Chingmei Formation up to Sungshan Formation, more and more sedimentary materials were derived from the Tertiary sedimentary province and the very low-grade metamorphic province in the southeastern, both of which are further from the Taipei Basin. Both the kaolinite content in Sungshan Formation and the montmorillonite content in Chingmei Formation gradually increase from the southern part to the northern part of the Taipei Basin, implying that more and more sediments were supplied from the northern volcanic region and the northwestern Linkou Tableland (Lin & Chen 1999).

In addition to these geological reviews, a more recent study was carried out at National Taiwan University of Science and Technology (Chin & Liu 1997). To understand the mineralogical properties of Taipei Clay, the X-ray diffraction and scanning electron microscope (SEM) were carried out. The samples used for the investigation were taken from Layer IV of K1 Zone. Figure 5 shows a typical result of

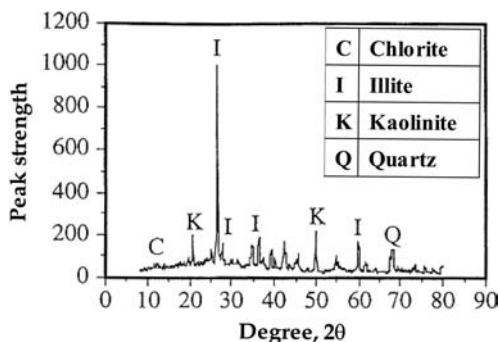
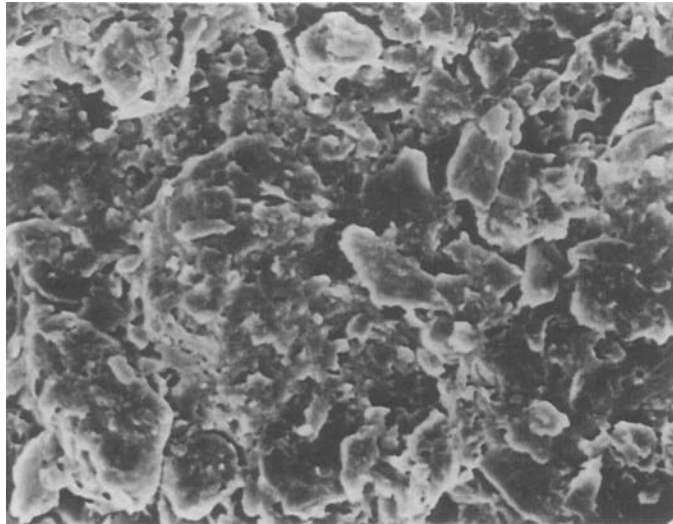


Figure 5. X-ray diffraction of clay at R-1 site (after Chin & Liu 1997).



(a) Horizontal view



(b) Vertical view

Figure 6. Microfabric of Taipei Clay viewing using SEM (after Chin & Liu 1997).

the clay, which suggests that the predominant clay mineral is illite (78%) and kaolinite (20%). Previous studies (e.g. Hung 1966) suggested that predominant clay minerals for the cohesive Sungshan Formation materials are illite and chlorite. The illite dominance in Taipei Clay is obvious. Microfabric of this K1 clay is shown in Figures 6a and 6b by the horizontal and vertical SEM results, respectively. The stack of illite plates can be more clearly seen in the vertical direction than in the horizontal view. In the horizontal direction, the arrangement of illite plates is somewhat random. For samples tested in this program, the natural water content is about 38%, the liquid limit and plasticity index are approximately 40% and 18%, and the specific gravity is about 2.73.

2.3 Groundwater and ground subsidence in the Taipei Basin

2.3.1 Hydrology of the Taipei Basin

Records of core logs and groundwater levels in the Taipei Basin indicated that the sediments between the ground surface and Chingmei Formation or the gravel materials at the bottom of Sungshan Formation constitute a regional confining bed. The confining bed consists mostly of silty and clayey materials, but it also contains several sand layers. The hydraulic head of these local sandy aquifers generally decreases

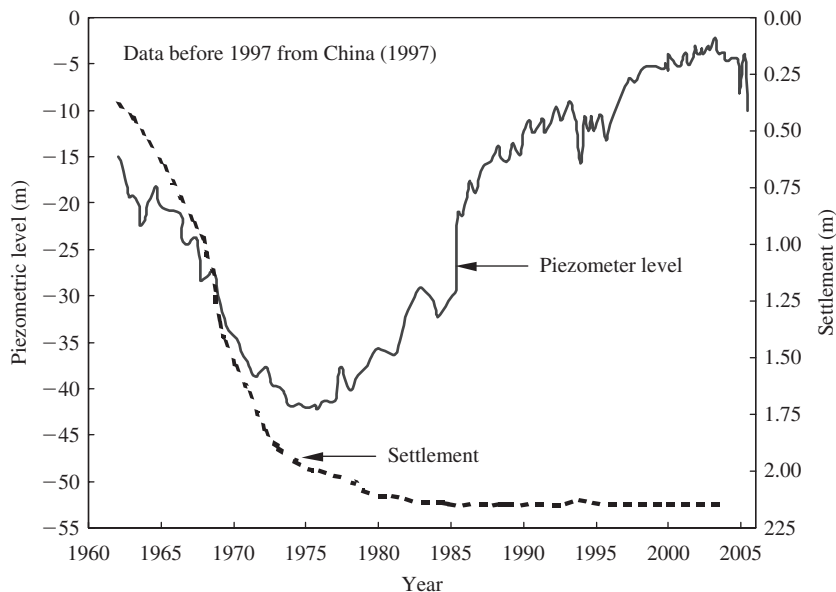


Figure 7. Groundwater drawdown and ground settlement in Taipei.

with the depth. Hsinchuang Formation, Chingmei Formation, and the gravel materials at the bottom of Sungshan Formation make up a regional aquifer. The aquifer is primarily composed of highly permeable gravel and sand bed. The major recharge areas are the river valleys of the Tahan River and the Hsintien River. A minor amount of recharge, however, is supplied by the downward flow through the overlying confining bed (Chia et al. 1999).

2.3.2 Groundwater pumping and ground subsidence

During the development of the modern city of Taipei, changes in the groundwater regime have been dominated by the extensive pumping from Chingmei Formation to supply water for the city. Similar to other cities that have experienced the same phenomenon, significant pumping-induced settlement occurred.

Figure 7 shows the typical water pressure drawdown in the Chingmei gravel and the associated settlement data for the downtown area (i.e. Taipei Main Station) of Taipei. Pumping of the groundwater from the Chingmei gravel began prior to 1960. Despite of restrictions imposed by the government in 1968, pumping from the Chingmei Formation continued until the mid-1970s, when the greatest drawdown was reached. Thereafter, there is a trend of recovery of water pressures in the gravel layer indicating the reduction in pumping. As shown in Figure 7, the ground surface settlement associated with pumping is closely correlated with the piezometric level within Chingmei Formation.

Another effect of water pressure reduction in the Chingmei gravel was the reduction in water pressures in the overlying Sungshan deposits. The effect of pressure reduction extended through the bottom three layers of Sungshan Formation (i.e. Layers I to III. Layer IV), which consists of silty clay, acted as an aquitard and essentially preserved hydrostatic conditions in the overlying materials. Figure 8 focuses on these changes of groundwater pressure profile within Sungshan Formation over the past 30 years. As shown in Figure 8, in 1980, prior to the TRTS construction, the groundwater pressure in the lower 4 layers of Sungshan Formation is significantly lower than the hydrostatic pressure. Measurements in late 1990's have shown consistent increases in the piezometric pressure that approaches toward the hydrostatic condition.

2.3.3 Groundwater recovery

As shown in Figures 7 and 8, the water pressures in the Chingmei gravel and the lower Sungshan deposits have been recovering since the mid-1970's. Figure 9 illustrates this groundwater recovery from 1974 to 1995 by including the typical water pressure data in the Chingmei gravel, Layer III of the Sungshan deposits, as well as measurements from Layer V. The data indicate a relatively slow rate of recovery

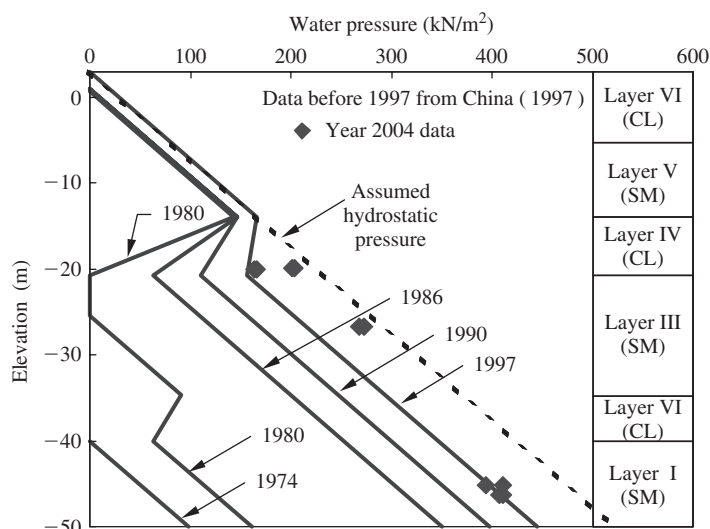


Figure 8. Groundwater pressure profile in the Taipei Basin.

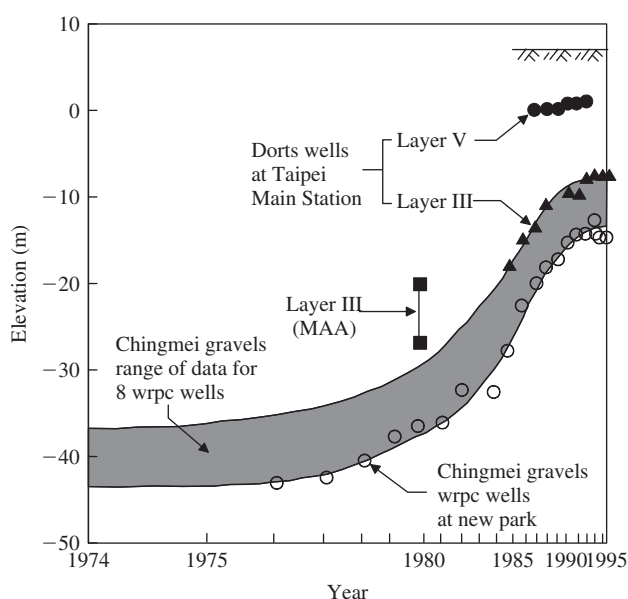


Figure 9. Piezometric level of Chingmei gravel in the Taipei Basin (after Chin 1997).

between 1976 and 1981. From 1981 to 1988, the recovery rate increases with an average head of +2 to 3 m/year. The recovery trend of Layer III closely follows the pressure within the Chingmei gravel while the piezometric head in Layer V is independent from the underlying aquifer. Despite of the consistent groundwater recovery, recent monitoring records indicate that the current groundwater pressure is still below the hydrostatic level (Figure 8). It should be noted that the slight drop in groundwater pressure in 1994 is due to large scale TRTS pumping from the Chingmei gravel. The TRTS construction dewatering reached its peak stage in 1993–1995 and exceeded the rate of natural recharge.

The past pumping with large reductions in groundwater pressure in the Taipei Basin have generally benefited previous excavations in the city. These excavations, typically for building basements, were well above the piezometric level in the lower Sungshan deposits; therefore groundwater control measures

were not required. The performance of these braced excavations was reasonably good in terms of wall deflections and ground movements; the favorable groundwater conditions undoubtedly contributed to the good performance.

For the TRTS excavations, groundwater control measures were required as the combination of greater excavation depth and the groundwater recovery places the excavation invert well below the piezometric level. Several construction failures occurred at TRTS sections which can be attributed to the higher groundwater pressure; therefore, the control of these water pressures is critical in achieving stability of the walls and limiting wall deflections and ground movements. Nevertheless, the past regional pumping also has some beneficial effects on the TRTS excavations. The past pumping in the Taipei Basin have caused the lower Sungshan deposits to become overconsolidated, thereby, reducing the compressibility of the cohesive layers to 10%–20% of that in the normally consolidated range. Since pumping associated with TRTS construction would caused less water pressure reduction in compressibility also translates into limited and acceptable level of surface settlements even with groundwater drawdown outside of the excavation (Chin 1997).

It should also be noted that the amount and the distribution of groundwater pumping can vary in the Basin. The monitoring results of this section are mainly from the area around the Taipei Main Station. Suburban areas, such as Nankang and Chingmei, can have different groundwater pressure distributions (Chin et al. 1991).

3 GEOTECHNICAL ZONING AND STRATIGRAPHY

3.1 *Geotechnical zoning*

In mid-1980's, geotechnical engineering mapping of Taipei City has been carried out. It is aimed at dividing the basin area of Taipei City into zones according to the depositional characteristics, stratifications and soil properties (MAA 1987). Huang et al. (1987) presented a summary on the methodology of this study and presents the results of the zoning.

In this study information on representative boring logs was collected from major projects within the city. More than seven hundred boring logs were compiled. Results of laboratory tests on samples taken from these boreholes were also reviewed and then included in the database. The zoning was achieved by the use of numerous fence diagrams, soil profiles, isopachous maps and isohyps maps of each sublayer and the surface of the bearing stratum. The isopachous map gives the isopachs which are lines of equal thickness of each sublayer. The isopachous maps provide a very clear idea about the variation of the thickness of each sublayer. The isohyps maps show the isohypses which are lines of equal altitude of the top of each layer. Since the elevation of the top of each sublayer can be obtained from corresponding isohyps maps, the drawing of geological cross section can be made fairly easily. With all these plots, the transformation of subsoils can be realized. The zoning was then made under the following two guidelines:

1. The drainage basins of the Tamshui River, the Keelung River, and the Hsintien River are the main regions of zoning.
2. The zones in each specific drainage basin are further divided according to distribution of the sublayers and the depositional environment.

The geotechnical zones were preliminarily delineated following these guidelines. Statistical analysis of the physical properties and engineering properties of the soils in each zone were then carried out. The zoning and statistical study process were iterated until a reasonable agreement was achieved.

Figure 2 shows the result of geotechnical zoning. Sungshan Formation can generally be classified into three major zones based on the drainage basin. Due to differences in location or depositional environment within the same drainage basin, each major zone can be further divided into 2 to 3 subzones. The relationship between the various zones and the main drainage basins is shown in Table 2.

The extent and characteristics of the strata in Zone T1, Zone T2, and Zone T3 are briefly described below:

- (1) Zone T1 – Zone T1 is the area located between the Tamshui River and the Hsintien River. Due to possible influence of shift of the river course, the variation between the layers is very complex. Layer VI is not distinct in most part of this zone and Layer V is about 15 m thick. Descriptive statistics of physical properties of T1 soils is given in Table 3.
- (2) Zone T2 – Zone T2 is the old downtown area of Taipei City, where is the area around the Taipei Main Station. The stratification characteristics of the six layers of Sungshan Formation in this zone are most

Table 2. Division of geotechnical zoning of the Taipei Basin.

Drainage basin	Geotechnical engineering zone
Tamshui river	T1, T2, T3
Keelung river	K1, K2
Hsintien river	H1, H2

Table 3. Descriptive statistics of physical properties of T1 soils (after MAA 1987).

Layer	Statistical items	Thickness (m)	γ_d (kN/m ³)	W_n (%)	G_s	w_l (%)	I_p (%)	Grain Size distribution (%)			
								Gravel	Sand	Silt	Clay
VI	mean	3.7	16.8	26.6	2.68	33.3	11.2	—	34	31	26
	COV	0.27	0.11	0.24	0.02	0.17	0.19	—	0.53	0.25	0.17
	n	7	10	10	8	10	9	10	10	7	7
V	mean	11.6	18.5	18.7	2.71	—	NP	11	63	16	5
	COV	0.25	0.12	0.38	0.004	—	—	1.49	0.21	0.51	1.14
	n	9	44	44	35	—	—	45	20	20	20
IV	mean	15.1	17.0	30.6	2.72	26.9	8.3	1	22	43	26
	COV	0.43	0.14	0.17	0.01	0.21	0.31	6.71	0.67	0.2	0.35
	n	9	45	45	19	42	25	45	45	28	28
III	mean	9.4	16.6	21.7	2.71	—	NP	1	54	18	6
	COV	0.52	0.05	0.16	0.004	—	—	5.83	0.47	0.45	0.61
	n	9	33	33	26	—	—	34	34	11	11
II	mean	12.9	15.1	28.5	2.73	31.2	9.4	—	—	—	—
	COV	0.58	0.05	0.10	0.004	0.19	0.34	—	—	—	—
	n	4	22	22	22	22	16	—	—	—	—
I	mean	—	—	—	—	—	—	—	—	—	—
	COV	—	—	—	—	—	—	—	—	—	—
	n	—	—	—	—	—	—	—	—	—	—

Note: clay particle size $<5 \mu$.

distinct. Generally speaking, the clayey soil is relatively thin and the thickness of Layer IV is about 5–10 m. The sandy soil layers are relatively thick and the thickness of both Layer V and III is about 15–20 m. The sandy soil layers become thicker and the clayey soil layers become thinner as it comes closer to the Tamshui River. Descriptive statistics of physical properties of T2 soils is given in Table 4.

- (3) Zone T3 – Zone T3 is located in the north of the Taipei Basin, close to the convergence of the Tamshui River and the Keelung River. The special characteristics of this Zone are relatively thick sandy soil and thinning out of the Layer VI.

The relatively thick layers of clayey soil and the relatively thin or thinned-out sandy soil strata are the characteristics of the drainage basin of the Keelung River. The thickness of Sungshan Formation in this region is directly influenced by the elevation of the underlying rock formation. The drainage basin of the Keelung River can be generally divided into two zones: Zone K1 and Zone K2.

- (1) Zone K1 – Zone K1 includes the Sungshan Airport and the newly developed city center area, where the Taipei City Government and Taipei Financial Center (Taipei 101) are located. In this zone, Layer V is found to be in absence. Layer III in these areas is also very thin or non-existing. Consequently, The standard six layers of Sungshan Formation is not applicable in this zone. The clayey soils from Layers II, IV, and VI become almost continuous and relatively thick. Descriptive statistics of physical properties of K1 soils is given in Table 5.
- (2) Zone K2 – Zone K2 is in the northeastern part of the Taipei Basin, which is located in the downhill of the Tatun Volcano Group. Similar to Zone K1, Zone K2 has a thick layer of clayey soil and lacks of the standard six-layer stratification. The average clay size particles content of the Layer VI and IV in Zone K2 is about 15% and 10%, respectively, which is higher than those in Zone K1. The average liquid limit and plasticity index of the Layer VI and IV are about 3 to 5% higher in Zone K2 as compared to those in Zone K1. The average liquid limit of Layer VI of this subzone reaches

Table 4. Descriptive statistics of physical properties of T2 soils (after MAA 1987).

Layer	Statistical items	Thickness (m)	γ_d (kN/m ³)	W_n (%)	G_s	w_l (%)	I_p (%)	Grain size distribution (%)			
								Gravel	Sand	Silt	Clay
VI	mean	4.5	14.5	31.2	2.72	35.8	12.9	–	10	58	32
	COV	0.46	0.06	0.14	0.01	0.17	0.37	–	0.74	0.18	0.36
	n	251	545	559	419	433	415	389	388	388	388
V	mean	10.1	15.4	26.3	2.68	–	NP	1	75	19	4
	COV	0.28	0.07	0.18	0.01	–	–	5.79	0.19	0.62	0.78
	n	259	1288	1316	996	–	–	950	950	950	950
IV	mean	9	14.3	32.1	2.72	34.3	12	–	8	61	31
	COV	0.63	0.06	0.13	0.01	0.15	0.37	–	1.06	0.15	0.38
	n	258	1441	1463	1071	1085	1064	1068	1068	1068	1068
III	mean	10.6	16.1	23.9	2.69	–	NP	–	60	34	7
	COV	0.55	0.06	0.18	0.01	–	–	–	0.29	0.82	0.69
	n	225	1116	1135	766	–	–	760	760	761	761
II	mean	8.4	15.5	27.2	2.72	30.3	9.2	0	9	67	25
	COV	0.57	0.06	0.15	0.004	0.16	0.36	–	0.81	0.12	0.36
	n	181	951	957	657	683	608	651	651	651	651
I	mean	4.5	17.0	20.3	2.69	–	NP	1	62	29	7
	COV	0.73	0.07	0.21	0.01	–	–	5.10	0.28	0.51	0.70
	n	112	273	281	183	–	–	184	184	184	184

Note: clay particle size $<5 \mu$.

Table 5. Descriptive statistics of physical properties of K1 soils (after MAA 1987).

Layer	Statistical items	Thickness (m)	γ_d (kN/m ³)	W_n (%)	G_s	w_l (%)	I_p (%)	Grain size distribution (%)			
								Gravel	Sand	Silt	Clay
VI	mean	4.3	14.3	32.1	2.71	34.1	11.1	0	11	61	29
	COV	0.7	0.08	0.17	0.01	0.18	0.43	–	0.74	0.1	0.42
	n	186	494	496	300	310	310	302	302	302	302
V	mean	6.6	17.0	20.1	2.68	–	NP	2	73	18	6
	COV	0.4	0.12	0.35	0.01	–	–	2.04	0.15	0.45	0.71
	n	152	461	474	285	–	230	230	230	230	230
IV	mean	20	13.7	35.1	2.72	35.2	12.9	0	4	61	35
	COV	0.29	0.07	0.14	0.004	0.15	0.35	–	1.13	0.15	0.31
	n	207	2557	2837	1474	1530	1488	1459	1457	1457	1457
III	mean	3.6	16.1	23.5	2.69	–	NP	0	54	36	11
	COV	1.15	0.07	0.18	0.01	–	–	–	0.30	0.37	0.56
	n	113	207	211	127	–	–	128	128	128	128
II	mean	9	15.2	28.3	2.71	33	12	0	12	54	35
	COV	0.63	0.09	0.22	0.01	0.2	0.42	–	0.97	0.23	0.41
	n	183	903	910	489	521	478	508	507	507	507
I	mean	4.7	16.8	20.9	2.68	–	NP	0	59	32	9
	COV	0.69	0.05	0.19	0.01	–	–	–	0.25	0.41	0.56
	n	91	233	245	128	–	–	124	124	124	124

Note: clay particle size $<5 \mu$.

67%, with average plasticity index of about 35%, and natural water content around 60%. Descriptive statistics of physical properties of K2 soils is given in Table 6.

According to the depositional environment, the Hsintien River Drainage Basin can be divided into two zones: Zone H1 and Zone H2.

(1) Zone H1 – It is located on the alluvial valley plain which is deposited by the Chingmei Creek, a tributary of the Hsintien River. The subsoil is mainly composed of clayey soil of about 20 to 30 m

Table 6. Descriptive statistics of physical properties of K2 soils (after MAA 1987).

Layer	Statistical items	Thickness (m)	γ_d (kN/m ³)	W_n (%)	G_s	w_l (%)	I_p (%)	Grain size distribution (%)			
								Gravel	Sand	Silt	Clay
VI	mean	4.8	12.7	40.6	2.73	38.9	15.6	0	3	53	44
	COV	0.57	0.07	0.14	0.1	0.16	0.32	—	1.21	0.19	0.26
	n	88	1508	1539	784	895	886	871	869	869	869
V	mean	4.9	14.1	33.1	2.67	—	NP	1	76	19	5
	COV	0.74	0.10	0.23	0.01	—	—	3.91	0.23	0.71	0.96
	n	53	188	198	119	—	—	125	126	125	125
IV	mean	26.9	12.8	40.5	2.73	38.8	15.5	0	3	53	45
	COV	0.22	0.08	0.14	0.01	1.01	0.32	—	1.23	0.19	0.39
	n	88	1654	1669	858	980	970	956	1495	954	955
III	mean	4.2	15.6	26.4	2.69	—	NP	3	60	26	10
	COV	0.61	0.12	0.30	0.01	—	—	1.98	0.26	0.46	0.72
	n	70	127	132	68	—	—	85	85	85	85
II	mean	7.4	14.3	31.2	2.70	33.7	12	0	9	58	33
	COV	0.95	0.08	0.2	0.01	0.19	0.41	—	1.05	0.18	0.33
	n	28	95	102	48	53	53	47	47	47	47
I	mean	3.9	16.6	21.2	2.70	—	NP	4	68	18	9
	COV	0.67	0.08	0.26	0.01	—	—	2.11	0.17	0.43	0.96
	n	15	20	22	10	—	—	10	10	10	10

Note: clay particle size $< 5 \mu$.

in thickness directly overlying rock formation. In areas along the Chingmei Creek, the subsoils are more variable and composed of interlayers of silty sand and silty clay. Within the silty sand stratum, gravel particles are sometimes found.

- (2) Zone H2 – The subsoils are mainly composed of sandy soil and gravel. The thickness of the clayey soil tends to increase from the Hsintien River eastwards. Descriptive statistics of physical properties of the H2 soils is given in Table 7.

3.2 Stratigraphy

Based on data collected during the construction of the priority network of the TRTS (Figure 2), the stratigraphy of the Taipei Basin was studied (Chin et al. 1994). It is noted that most of the deeper excavations for the TRTS projects is located in the T2 and K1 Zones. The stratigraphy in the T2 zone is relatively uniform, consisting of six alternating layers of clays and sands. The uppermost Layer VI together with Layers IV and II are cohesive soils, while Layers V and III are silty sands. The lowermost Layer I is variable and contains both clayey and sandy materials. In the K1 Zone, sandy layers are thin or absent and the profile is dominated by cohesive soils.

A typical east-west section (Figure 10) along the TRTS Panchiao-Nankang Line (from Panchiao to Nankang) illustrates the general change in stratigraphy from alternating clayey/sandy layers in the T2 Zone to the thick clay deposits in the K1 Zone to the east. Figure 11 shows a typical north-south section along the TRTS Tamsui-Hsintien Line (from Peitou to Hsintien). The northern section of the line is in K2 Zone, where all soil deposits are cohesive soils. Typical T2 stratigraphy occurs in the center section of this line. A promontory of sandstone and tuff abruptly delineates the T2 Zone from the gravelly H2 Zone that occupies the southern section of the line.

For the TRTS projects, Layer IV of the Sungshan deposits frequently occurs at about the invert of open excavations and is the medium through which tunnels will pass in the T2 zones. In area nearby the Taipei Main Station, this material is silty with low plasticity. To the east, it becomes more clayey and construction performance in this area was dominated by deep deposits of soft clay. The strength of these Layer IV materials has hence become focus of studies.

4 STATE AND YIELD BEHAVIOR OF TAIPEI CLAY

To investigate state and engineering properties of Taipei Clay, a Designated Task was carried out as part of the GESC project of TRTS. Results obtained from this Designated Task provided principle materials

Table 7. Descriptive statistics of physical properties of H2 soils (after MAA 1987).

Layer	Statistical items	Thickness (m)	γ_d (kN/m ³)	W_n (%)	G_s	w_l (%)	I_p (%)	Grain size distribution (%)			
								Gravel	Sand	Silt	Clay
VI	mean	5.6	13.9	33.0	2.71	35	13.8	0	8	57	35
	COV	1.12	0.06	0.18	0.01	0.16	0.34	—	1.08	0.14	0.27
	n	5	32	33	20	22	23	22	22	22	22
V	mean	14.6	19.1	13.3	2.69	—	NP	22	52	20	5
	COV	0.21	0.13	0.54	0.01	—	—	1	0.27	0.64	0.62
	n	9	55	65	41	—	—	42	42	42	42
IV	mean	6.7	14.6	31.0	2.71	31.2	10.1	0	10	65	25
	COV	0.86	0.07	0.16	0.01	0.16	0.31	—	0.79	0.12	0.40
	n	10	61	62	37	26	42	41	41	41	41
III	mean	6.3	16.9	20.9	2.70	—	NP	2	68	25	7
	COV	0.5	0.08	0.16	0.01	—	—	4.12	0.26	0.57	0.82
	n	7	29	30	14	—	—	17	15	15	15
II	mean	—	14.4	30.8	2.71	33.2	9.5	0	16	66	18
	COV	—	0.02	0.11	0.004	0.15	0.07	—	0.66	0.21	0.55
	n	4	8	9	6	7	6	7	7	7	7
I	mean	—	17.2	21.3	2.70	—	—	0	80	18	2
	COV	—	—	—	—	—	—	—	—	—	—
	n	1	2	2	1	—	—	1	1	1	1

Note: clay particle size $< 5 \mu$.

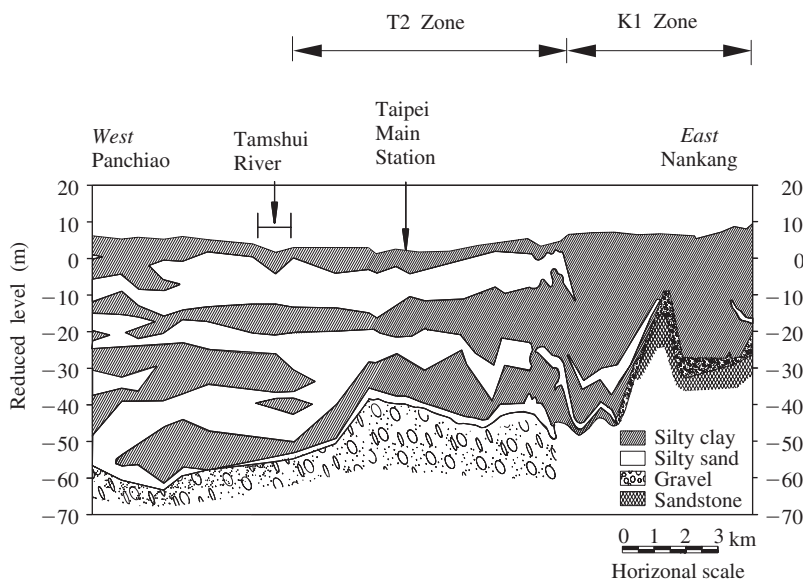


Figure 10. Stratigraphy along the E-W Nankang Line (adapted from Chin et al. 1994).

to delineate engineering characteristics of the clay in this paper. In this section, the state properties of Taipei Clay is presented primarily based on these results. To investigate the cohesive Sungshan deposits, detailed and careful borings were put down at locations shown in Figure 2. Borehole R-1 is located in the K1 Zone and Borehole R-2 is located in the T2 Zone, while Borehole R-3 is selected to be in transition area of K1 and T2 Zones. These test sites were chosen to reflect the varying nature of stratigraphy across the Basin, as suggested by Figure 10. At these sites, stationary piston sampling was carried out continuously to retrieve undisturbed samples for laboratory testing. During drilling, the mud in the borehole was controlled to have a unit weight of between 10.5 kN/m³ and 11.0 kN/m³ to reduce disturbance resulted from swelling. At the K1 site, a section of samples were sent to Golder Calgary for

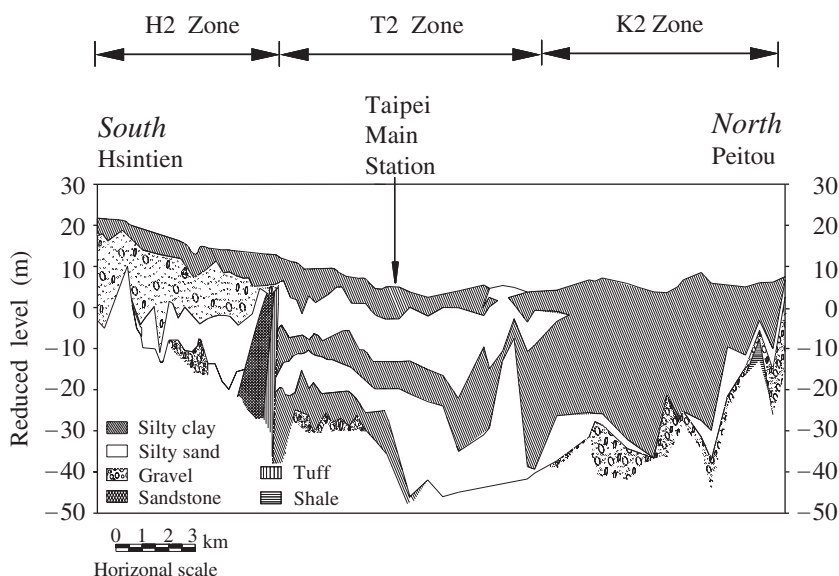


Figure 11. Stratigraphy along the N-S Hsintien Line (adapted from Chin et al. 1994).

studying the state and yield surface of Taipei Clay. A portion of undisturbed tube samples from depth 18 m to 22 m were sent to MIT for a special laboratory testing program to study stress-strain-strength behavior of the clay (Germaine 1992). All the other tests were performed at MAA laboratory in Taipei. Results of this Designated Task have been summarized in Chin et al. (1994), Hu et al. (1996), and Chin & Liu (1997).

4.1 State of Taipei Clay

To understand the fundamental strength properties, the concept of state has been used to evaluate behavior of the cohesive Sungshan deposits (Chin et al. 1994). The strength behavior of a soil is controlled in a first order sense by its void ratio and the effective stresses to which it is subjected. The current state of the soil is represented as a point on a void ratio – effective stress plot. Current state is quantified by its distance from a reference line. For sands, the steady state line (SSL) is used as a reference, and the current state is quantified as the void ratio difference between the current state and the SSL at the same stress level (i.e. state parameter Ψ). Details of the basic state parameter concept is referred to Been & Jefferies (1985). Chen et al. (1993) presents the results of the study on state parameters and related engineering properties of Taipei silty sand. For clays, the virgin consolidation line (VCL) is used as a reference, and current state is quantified as the ratio of the current stress to the maximum past pressure (OCR). The use of OCR to represent the state of clays is the basis of the SHANSEP (Stress History and Normalized Soil Engineering Properties) concept (Ladd & Foott 1974).

Since the state of clays is defined in terms of OCR, accurate measurements of the following effective stresses are required:

(1) Vertical in-situ stress (σ'_{v0})

This is more difficult in Taipei than is often the case because of the current complex groundwater conditions caused by past pumping from the Chingmei gravels and incomplete recovery of water pressures since pumping stopped. Total stresses are readily calculable but piezometers are required to define the in-situ water pressure. An estimate of σ'_{v0} can also be obtained from conventional oedometer tests using the “work/unit volume” approach proposed by Becker et al. (1987).

(2) Vertical yield stress (σ'_{vy})

These values are estimated from the results of oedometer tests. However, for the cohesive Sungshan deposits, the $e - \log$ stress curves from oedometer tests are rounded with indistinct yield points. Hence, interpretation of these data is best carried out using the “work/unit volume” approach.

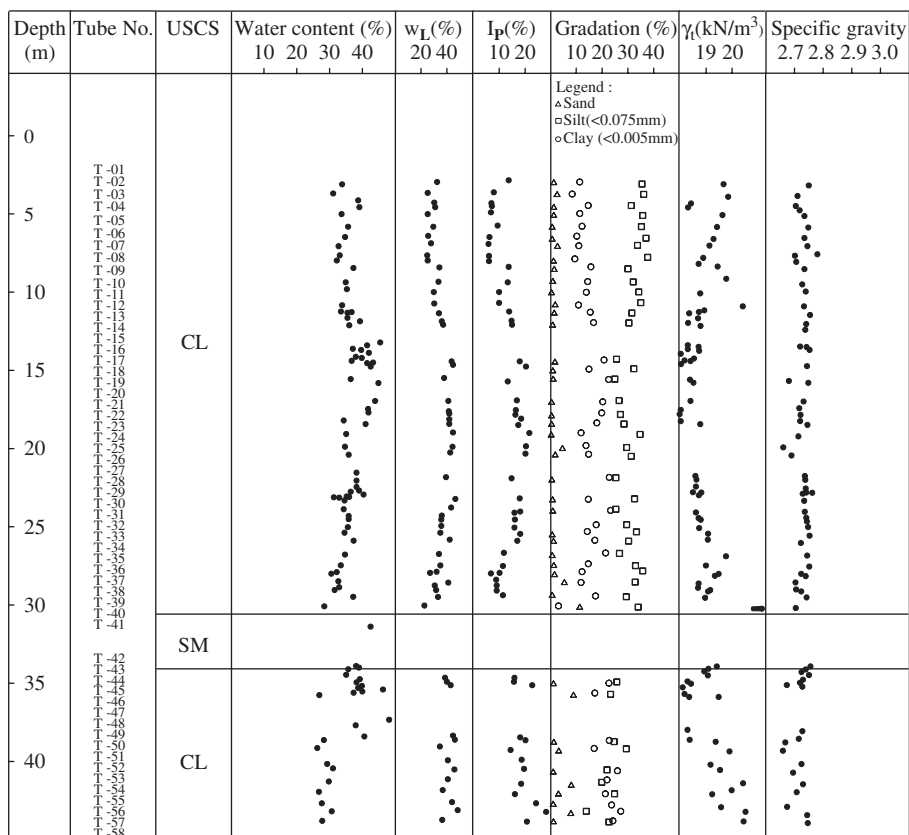


Figure 12. Index properties of K1 soils (after Chin et al. 1994).

(3) Horizontal in-situ and yield stresses (σ'_{ho} and σ'_{hy})

These values are estimated using the “work/unit volume” interpretation of the results of oedometer tests carried out on vertically trimmed samples.

4.1.1 K1 zone

Figure 12 shows the soil profile at the location of Borehole R-1, which reflects the typical strata patterns as expected for the K1 Zone. Now the location of the Borehole R-1 is the Taipei City Hall Station of TRTS Nankang Line. At this site, the soil is predominated by silty clay except for a 4-m thick silty sand at about 30 m within the first 45 m of deposits below the ground surface. The nature of the clayey soil changes slightly but consistently with depth, where the clay content increases down to about 15 m and then decreases again below this depth. This variation is reflected in the index properties shown in Figure 12.

Piezometric pressures were determined from inspection of piezometers installed in the boring and in other borings drilled previously at nearby locations. While the latter show some variation with time, a reasonably definitive linear trend with depth can be drawn from the most likely values. No information is available above a depth of 15 m, and it is assumed that the current piezometric profile continues upward until it intersects the hydrostatic line associated with a groundwater level at a depth of 1 m. The current piezometric pressure profile is shown in Figure 13 by the solid line, and this would represent the upper bound of piezometric pressure.

Based on the measured piezometric profile and calculated total stresses, the current in-situ vertical effective stress (σ'_{vo}) profile can be accurately determined, as shown in Figure 13. The σ'_{vo} values estimated from vertically loaded oedometer tests are reasonably consistent with the calculated σ'_{vo} profile. The profiles of vertical and horizontal effective yield stress (σ'_{vy} and σ'_{hy}) and other state parameters (K_o and OCR) with depth also are shown in Figure 13. Above 15 m depth, the σ'_{vy} values are approximately

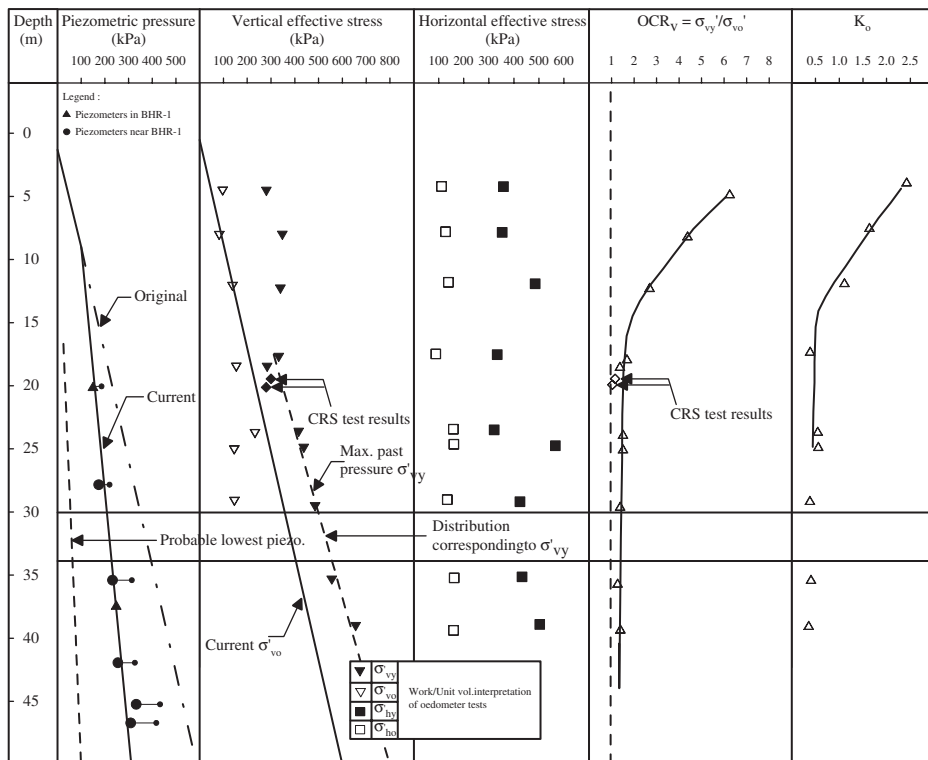


Figure 13. State properties of K1 soils (after Chin et al. 1994).

constant but are relatively high given the depth. The σ'_{hy} values are as high as σ'_{vy} . OCR_v values also are high at shallow depth, but it decreases to about 1.5–2 at about 15 m deep. Similar trend also is seen for the K_o profile, which has high values above 15 m depth.

The state data below 15 m can be readily understood in terms of known stress history of the deposit. Therefore, the σ'_{vy} values form a consistent trend and lie above the current σ'_{vo} profile, and the magnitude of overconsolidation is consistent with a past piezometric profile that reflects a 35–40 m water level lowering in the underlying Chingmei gravels. As a result, OCR decreases slightly from about 1.7–1.4 with depth. Given the stress history below 15 m depth, the K_o values that range from 0.45 to 0.66 appear reasonable. Nevertheless, unrealistically low values were measured at depths and have been ignored.

4.1.2 T2 Zone

The stratigraphy and index property data of Borehole R-2 are shown in Figure 14. Borehole R-2 soil profile is typical of the T2 Zone, where all of Layers I to IV can be easily recognized. The piezometric pressure profile and other state properties are given in Figure 15. The piezometric conditions in the depth range for Layers IV to VI are well defined based on the two piezometers installed in Layer IV and the piezometric pressures measured in Layer V at the end of a companion CPT dissipation test. It is found that the measured piezometric profile is consistent with other data in the general T2 Zone. Based on these data, the state characteristics of the two silty clay layers can be determined.

The 3 m thick Layer VI silty clay is moderately overconsolidated based on the oedometer test results. However, the degree of overconsolidation is not as high as in the upper portion of the K1 Zone.

Layer IV consists of a moderately plastic clayey silt to silty clay material that is typical throughout the T2 zone. Two sets of oedometer tests indicated that this layer is slightly overconsolidated with an OCR of 1.6. The magnitude of overconsolidation above the current σ'_{vo} profile is consistent with the fact that the piezometric pressure at the top of the underlying sandy layer was zero during the period of maximum groundwater lowering when pumping took place from the lower Chingmei gravels. In addition, the σ'_{ho} values from the horizontally loaded oedometer tests provide reasonable K_o values of about 0.5.

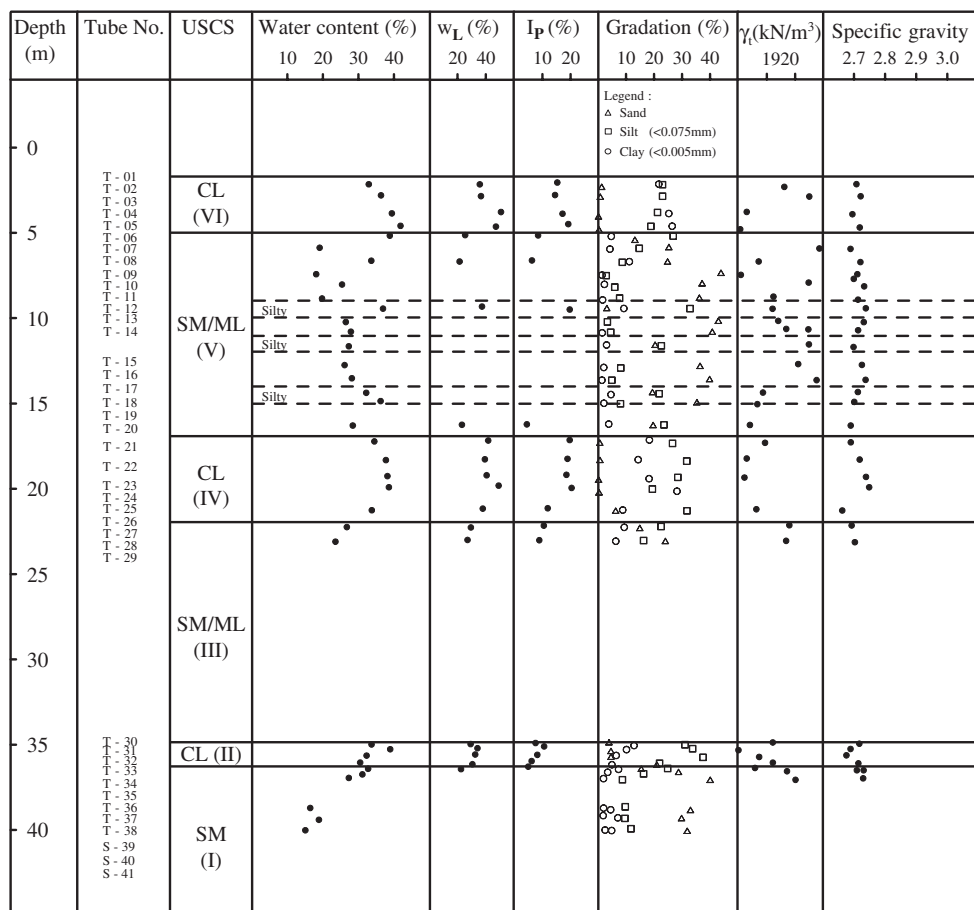


Figure 14. Index properties of T2 soils (after Chin et al. 1994).

4.1.3 *K1/T2 transition zone*

The stratigraphy and index property data of Borehole R-3 are shown in Figure 16. Borehole R-3 is located at the transition of T2 and K1 zones. The typical six-layer Sungshan deposit profile is still present but with thick cohesive layers compared to that in the T2 zone. In addition, Layers I to III at this location are poorly defined below 29 m in depth, and this zone is best described as silt – clayey silt with silty sand interbeds.

The piezometric profile of this test site is shown in Figure 17, and it is reasonably well defined by piezometers installed in this borehole and others previously installed in nearby locations. The results are very similar to those at the R-1 site. For the state characteristics, Layer VI is over-consolidated with high K_o values as is the case for the K1 location. Furthermore, for Layer IV clay, six pairs of vertically and horizontally loaded oedometer tests were conducted, and the results provide a good definition of the state of soils within this layer. As can be seen in Figure 17, the σ'_{vy} profile is reasonably consistent with the maximum vertical effective stress calculated based on the maximum past groundwater lowering due to pumping from the Chingmei gravels. The resulting OCR is approximately consistent at about 1.4–1.6, and K_o values are consistently equal to about 0.6.

4.2 Yield behavior

Most cohesive soils exhibit both “small strain” and “large strain” behavior. Small strain behavior is characterized by undrained strength in compression and to a lesser extent, in extension. A common example of small strain behavior in a drained test is the yield stress evident in an oedometer test. Large

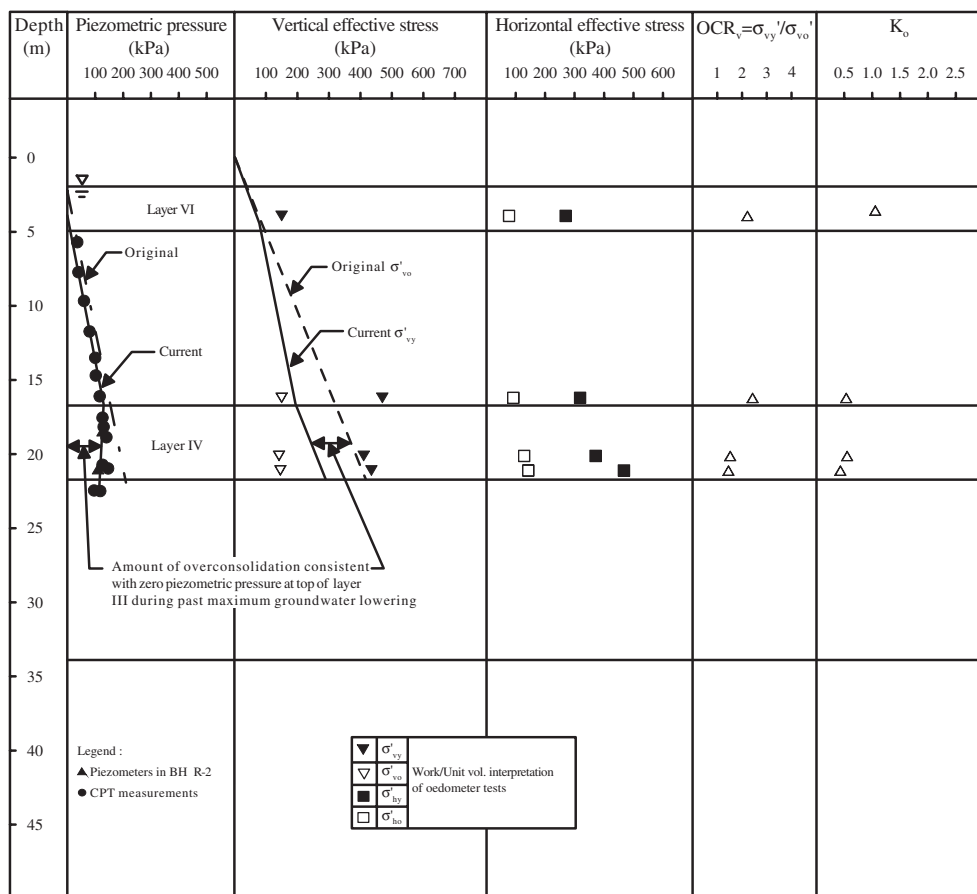


Figure 15. State properties of T2 soils (after Chin et al. 1994).

strain behavior occurs in undrained strength tests when the material state reaches the ϕ' line. In an oedometer test, large strain behaviour occurs at stresses in excess of the yield stress (i.e. the normally consolidated stress range).

The total stress paths imposed in triaxial strength tests and oedometer tests are limited by the test conditions. However, under field loading conditions, the soil can be loaded along a wide variety of stress paths depending on the nature of loading. While the response of the soil to varying stress paths will vary in detail, there is a distinction between small and large strain behavior regardless of stress path.

Small strain behavior under various stress paths can be investigated in the laboratory by carrying out drained triaxial tests along various stress paths. For each stress path in q - p' stress space, a yield point can be identified which indicates the boundary between small strain and large strain behavior. The locus of these yield points in q - p' stress space is termed the yield envelope. Small strain behavior occurs at stress states within the yield envelope, while large strain behavior occurs outside the yield envelope.

The undrained compression strength will coincide with the upper surface of the yield envelope while the extension strength will coincide with the lower surface of the yield envelope. The apex of the yield envelope is typically defined by the preconsolidation pressure and the axis of the envelope will be approximately symmetrical around the K_o line. Except for bonded or highly structured soils, the large strain lines in compression and extension will define the upper and lower limits of the yield envelope at lower stress levels; by definition stress states beyond the ϕ' lines in compression and extension cannot exist.

The yield envelope concept represents a simple qualitative stress-strain-strength model for lightly overconsolidated soils. It is useful for understanding the behaviour of soils under field loading (Folkes & Crooks 1985, Crooks et al. 1984). It is also useful for evaluating the consistency of laboratory and field strength data.

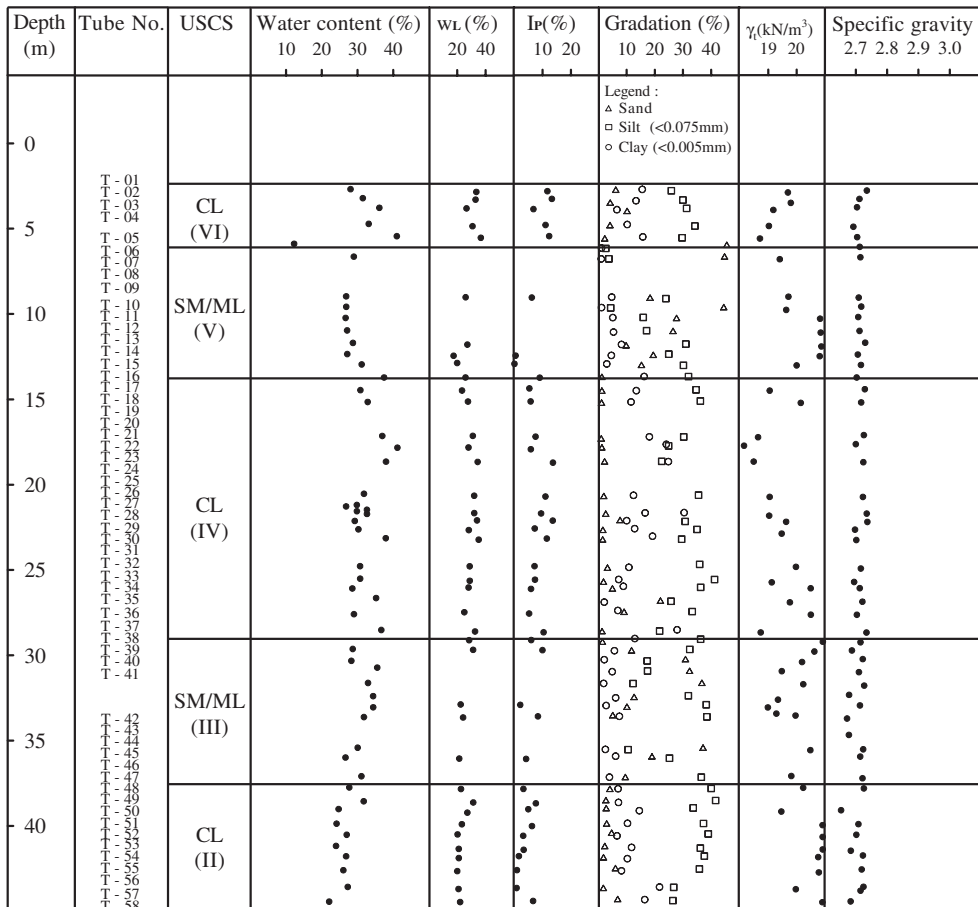


Figure 16. Index properties of K1/T2 soil (after Chin et al. 1994).

4.2.1 Yield envelopes for cohesive Sungshan deposits

The yield envelopes for samples of the cohesive Layer IV Sungshan deposits from each of the three locations were determined in drained triaxial tests. The test samples were reconsolidated to their in-situ stresses prior to testing. Because the samples were obtained from different depths, the yield data were normalized with respect to their respective in-situ vertical yield stresses. The yield data from these tests are shown on Figure 18 and describe a consistent generalized yield envelope for the Layer IV deposits. Data from other tests also fit well with the generalized yield envelope:

- The results indicate a consistent yield behavior regardless of sample origin (i.e. the normalized yield envelope for each material type is essentially the same).
- The compression and extension strengths predicted based on normalized (SHANSEP) strength behavior using a typical OCR value of 1.6 correlate closely to the upper and lower portions of the generalized yield envelope.
- The apex of the generalized yield envelope is reasonably well defined by the yield stress measured in oedometer tests and is consistent with a K_o value of 0.55.
- The large strain effective stress lines clearly provide upper and lower limits for the generalized yield envelope.

5 ENGINEERING PROPERTIES OF TAIPEI CLAY

To understand the fundamental behavior of Taipei Clay, high quality continuous piston samples were taken from K1 zone for advanced laboratory tests in the Designated Task of GESC, TRTS. In that study,

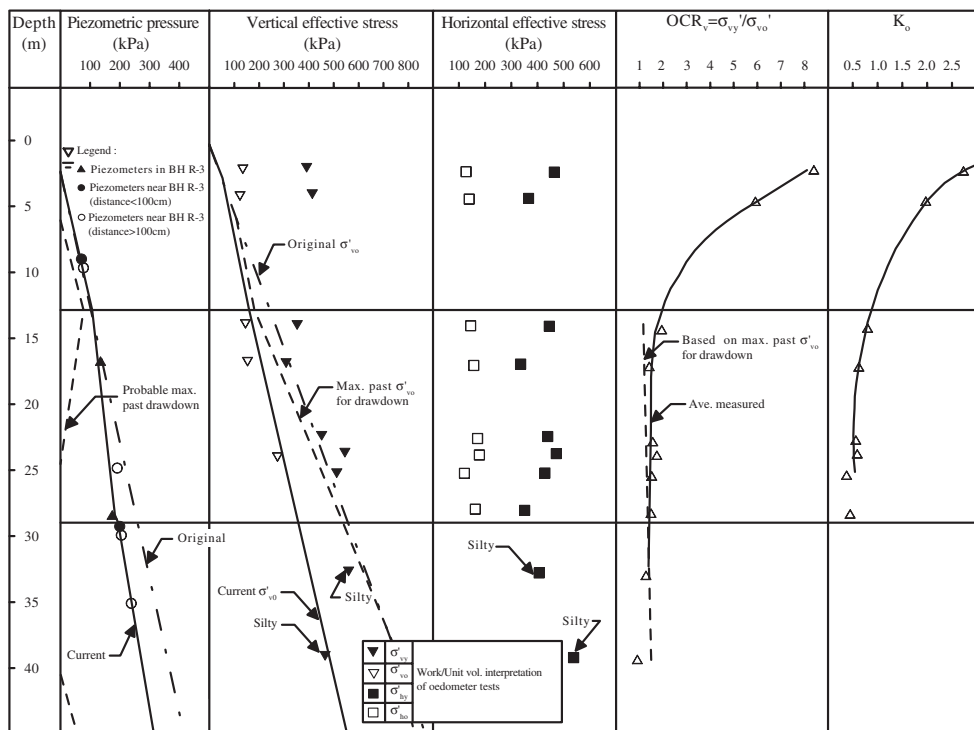


Figure 17. State properties of K1/T2 soil (after Chin et al. 1994).

constant-rate-of-strain (CRS) consolidation tests were conducted to study the volumetric behavior under normally and over-consolidated states. K_0 -consolidated undrained triaxial compression, direct simple shear, and triaxial compression tests were conducted using SHANSEP procedures at various OCRs. These test results provide insightful information on undrained behavior of Taipei Clay. Along with the other studies, engineering properties of Taipei Clay are discussed in this section.

5.1 Sample preparation and test procedures

This section presents a summary of sample preparation procedures at the Geotechnical Laboratory at MIT (Germaine 1992). The testing program conducted at MIT also are summarized in this section.

5.1.1 Sample preparation

For the special laboratory testing program, four 10 cm (3 inch) diameter by 76.2 cm (30 inch) long Shelby tube samples taken from Borehole R-1 were sent to the Geotechnical Laboratory at MIT for compressibility and strength tests. The specimens were first examined using radiography and the best quality; most uniform sections were selected for preparing test specimens. Figure 19 shows an example of the X-ray examination film (Germaine 1992). Specimens were prepared for all engineering tests following very careful procedures, and those of best quality were selected for CRS tests. The tubes were cut above and below the selected specimens to reduce disturbance due to extrusion. The remaining segments of the tube were sealed for later use. The specimen was then extruded from the tube using the following procedures:

1. A fine wire was penetrated through the soil along the inside of the tube.
2. The wire was used to cut the soil free from the inside perimeter of the tube
3. The soil was gently pushed by hand free of the tube.

The resulting block of soil was trimmed in different manners depending on the specific type of tests. In this study, a total of six triaxial, one direct simple shear, and two CRS consolidation tests were performed.

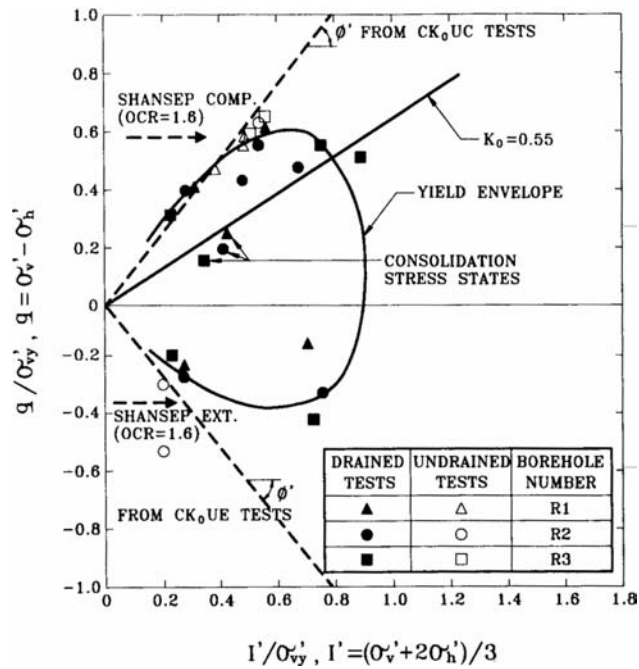


Figure 18. Yield envelope for Layer IV soils (after Chin et al. 1994).

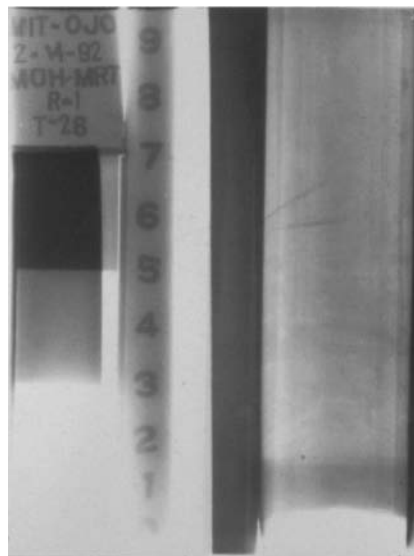


Figure 19. X-ray examination result of R-1 sample (after Germaine 1992).

5.1.2 Testing program

CRS consolidation tests were performed using MIT device that was developed by Wissa (1971). However, modification to the original apparatus was made by removing the inner diaphragm seal to reduce the potential for friction errors. The specimen is 5 cm in diameter and 2 cm in height. After assembling the test equipment, the specimen was saturated by backpressure at constant volume to about four atmospheres (390 kN/m²). Then consolidation was performed by adding vertical pressure at a fixed strain rate

Table 8. Summary of consolidation tests of Taipei Clay (after Chin & Liu 1997).

Depth (m)	Test no.	w _n (%)	e _o	u _b (kPa)	ε (sec ⁻¹)	σ' _{vo} (kPa)	CR	RR	c _v (cm ² /sec)	σ' _{vm} (kPa)
19.90~19.96	CRS66	38	1.07	393	1.4 × 10 ⁻⁶	193	0.175	—	3.7 × 10 ⁻³ ~6.3 × 10 ⁻³	280
19.35~19.38	CRS68	40	1.10	392	1.44 × 10 ⁻⁶	188	0.180	0.02	1.5 × 10 ⁻³ ~2.3 × 10 ⁻³	300
18.50~18.54	OED44	42	1.17	—	—	181	0.183	—	1.3 × 10 ⁻³	240

of about 0.5%/hr. Under normal condition, the ratio of the measured base pore water pressure over the vertical stress shall not exceed 0.05.

Triaxial tests were performed using a computer controlled system and procedures developed at MIT. The triaxial cell features a low friction rolling diaphragm seal, a fixed top cap geometry and double drainage. The test specimen is about 8 cm tall and 10 cm² in cross section. All measurements were made externally using electronic transducers. The specimen is enclosed in two prophylactic membranes. Radial drainage is provided by vertical filter drains for compression tests and spiral drains for extension tests. Silicone oil was used for the cell fluid to prevent membrane leakage. Data were corrected for membrane resistance and filter drain resistance in compression tests. For the six triaxial tests, SHANSEP procedures were followed. Undrained tests with OCRs = 1, 2, and 4 were performed. The specimen was consolidated to the desired maximum stress state (usually 2 to 4 times the estimated maximum past pressure) using a constant rate of deformation at about 0.07 %/hr and controlled by computer to maintain the K_o condition or a specific stress path. The maximum stress is maintained for 24 hours. When appropriate consolidation was done, the specimen was rebounded to the desire OCR and controlled by computer to maintain the final stress for at least 8 hours. Before shearing, drainage lines were closed and the change in pore water pressure was monitored for 30 minutes to confirm that the specimen is sealed and the rate of secondary compression is small. Finally, the specimen was sheared at a constant rate of axial deformation ranging from 0.2%/hr to 0.5%/hr to failure. This rate was estimated to be slow enough such that dilation will not occur during shearing.

The basic Geonor direct simple shear device was used at MIT, and the device is fully automated to allow precise control during both consolidation and shearing phase of the test. The specimen is prepared to have a cross section area of 35 cm² and 2 cm height. SHANSEP type of procedures were followed for the testing. The specimen was consolidated at a constant rate of axial deformation of about 0.5 %/hr to a predetermined maximum vertical consolidation stress, and the maximum stress is held constant for about 24 hours or one cycle of secondary compression. Undrained shear was performed at a constant rate of horizontal deformation of about 5%/hr. Undrained conditions are maintained by computer control of the vertical stress to ensure constant specimen, and the computer automatically adjusts for the compliance of the apparatus.

5.2 Compressibility and permeability

Two K1 Layer IV samples were tested under CRS consolidation tests to study the compressibility and permeability characteristics of Taipei Clay (Germaine 1992). In addition, another sample was tested using conventional oedometer apparatus (Chin & Liu 1997). The summary of these consolidation tests is given in Table 8, and Figure 20 shows the stress–strain relationship of these tests.

5.2.1 Compressibility

Figure 19 shows very consistent CRS consolidation test results for the clay, and it also shows that the yield point (or the maximum past consolidation stress, σ'_{vm}) can be clearly depicted. Both CRS test results also shows a linear normal consolidation line, which suggests that soil particles were not deflected and the clay is not highly structured. Therefore, the compressibility during normal consolidation can be represented by a single compression index (C_c), and the values for these two CRS tests are 0.36 and 0.38. Since compressibility of soil is a function of the void ratio, the compression ratio [CR = C_c/(1 + e_o)] is a better index for comparison. The CR value for the Taipei obtained from the CRS test is about 0.18, which is very comparable to that of the Boston Blue Clay (CR = 0.17, Kavvasdas 1982).

The compressibility of clay under overconsolidation condition can be analyzed through results of rebound and recompression loadings. Figure 20 demonstrate that hysteretic behavior during rebound

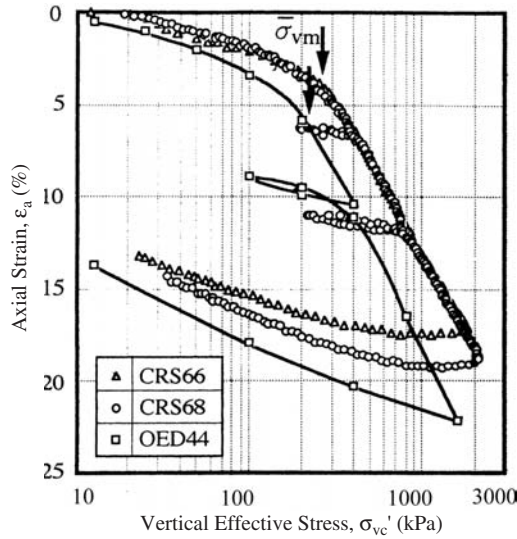


Figure 20. Stress–strain relationship for Taipei Clay under CRS and oedometer tests (after Chin & Liu 1997).

and recompression exists for Taipei Clay. In traditional soil mechanics, the rebound and recompression behavior is related to OCR. However, if the true hysteretic response is to be considered, it is not enough using OCR to describe the rebound and recompression behavior at the same time. Instead, the concept of stress ratio has to be introduced (Whittle et al. 1993). The stress ratio (ξ) is defined as follows:

$$\xi = \sigma'_o / \sigma'_c \text{ (for rebound)} \quad (1)$$

$$\xi = \sigma'_c / \sigma'_o \text{ (for recompression)} \quad (2)$$

in which σ'_c is the rebound or recompression stress that the soil experienced and σ'_o is the consolidation stress at the beginning of the rebound or recompression process. Based on these definition, ξ is equal to OCR during rebound, however it is only a dimensionless index during recompression. Nevertheless, this stress ratio concept is important for establishing a comprehensive soil model.

The incremental vertical strain during both rebound and recompression stages are plotted versus the stress ratio in Figure 21 (Whittle et al. 1993). It is noted that the incremental strain has been corrected for the amount induced by the secondary compression in this plot. As can be seen from this plot, values of swelling ratio (SR) and rebound ratio (RR) are not constant, but increase with increasing ξ . It is also evident that SR and RR values for both rebound and recompression stages are very close, and this suggests that the hysteretic loop is symmetrical. When the value of ξ is between 1 and 3.5, both SR and RR values are approximately smaller than 0.02. In addition, the overconsolidation compressibility of the Taipei Clay is very close to the Boston Blue Clay, as shown in Figure 21.

The results of CRS tests can be compared to that of the traditional oedometer test. For the maximum past consolidation stress (σ'_{vm}), results obtained from the CRS tests are approximately 280~300 kPa. The σ'_{vm} value is about 240 kPa for the oedometer test, which is about 14% to 20% smaller than that for the CRS tests. Leroueil et al. (1983) showed that for soils taken from the same depth, values of σ'_{vm} are higher for tests under higher strain rates, and he concluded that the σ'_{vm} value obtained using the CRS test is approximately 13% ($\pm 9\%$) greater than that obtained by the oedometer test. With reference to this study, results for Taipei Clay are quite reasonable. For the compression index, C_c (OED) is about 0.4, while C_c (CRS) ranges from 0.36 to 0.38. However, in terms of CR, all the results are very comparable, and the value is about 0.18. Under overconsolidation condition, the compressibility suggested by the oedometer test is very close to that of the CRS test, as shown in Figure 20.

5.2.2 Permeability

The vertical coefficient of permeability (k_v) from the results of CRS tests can be computed based on suggestions given by Wissa et al. (1971). The computed vertical coefficient of permeability from these two CRS tests are plotted versus the void ratio in Figure 22. The estimated value of in-situ k_v

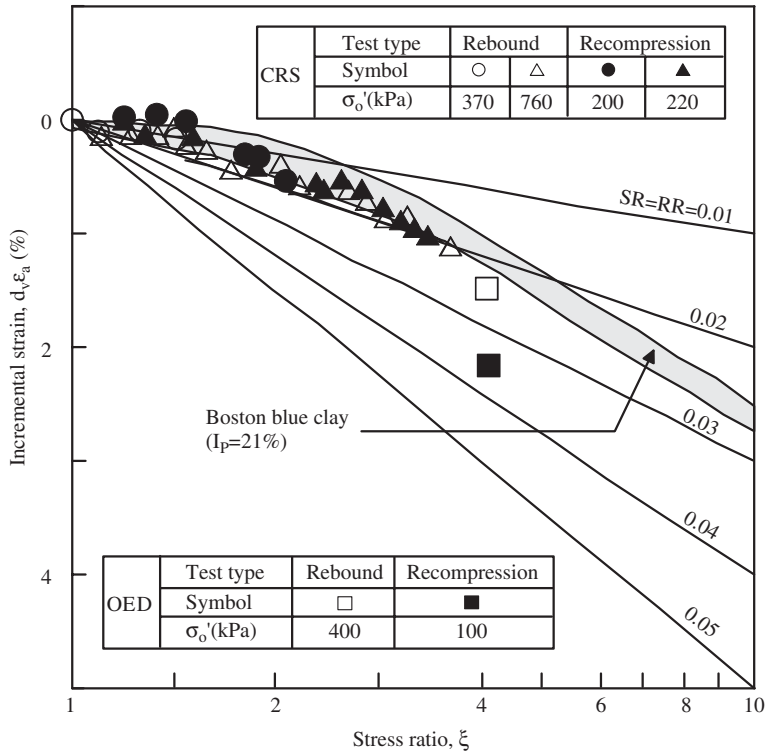


Figure 21. Compressibility of Taipei Clay under overconsolidation (after Whittle et al. 1993).

is about 0.6 to 1.2×10^{-7} cm/sec. These values are consistent with results from multi-stage constant head triaxial permeability tests reported by Whittle et al. (1993), which showed that k_v values of Taipei Clay at Borehole R-1 range from 1 to 2×10^{-7} cm/sec. As can be seen, there is an approximate linear relationship between $\log k$ and the void ratio. In general, the coefficient of permeability decreases with decreasing void ratio. From these two tests, values of the slope (C_k) of the e - $\log k$ lines are 0.55 and 0.58 , respectively. Tavenas et al. (1983) studied the permeability of various clays and suggested that C_k can be expressed by the equation given below:

$$C_k = 0.5 e_o \quad (3)$$

in which e_o is the initial void ratio. The estimated values of C_k using Equation 3 for the two CRS tests are 0.54 and 0.55 , which are very close to the measured results.

5.3 Undrained shear behavior

5.3.1 Normalized behavior

Based on a series of CIUC tests, it is found that the undrained shear strength of Taipei Clay can be described by the SHANSEP equations. Moreover, the normalized undrained shear strength (su/σ'_c) of Taipei Clay is a function of plasticity index (I_p), and the results are shown in Figure 23. Chin et al. (1989) also summarized effects of anisotropy and shearing modes on the undrained shear strength for Taipei Clay based on tests conducted at MAA, and the results are shown in Figure 24. It should be noted that the predetermined K_o was used for these CAUC and CAUE tests.

5.3.2 K_o -triaxial compression and extension tests

Strength tests of the Designated Tasks carried out at MIT included six undrained triaxial shear tests. In all of the triaxial tests, specimens were consolidated to various OCRs ($OCR's = 1, 2$, and 4). The K_o -consolidated specimens then were sheared in compression and extension modes (CKoUC and CKoUE). The basic information of these six triaxial shear tests is tabulated in Table 9.

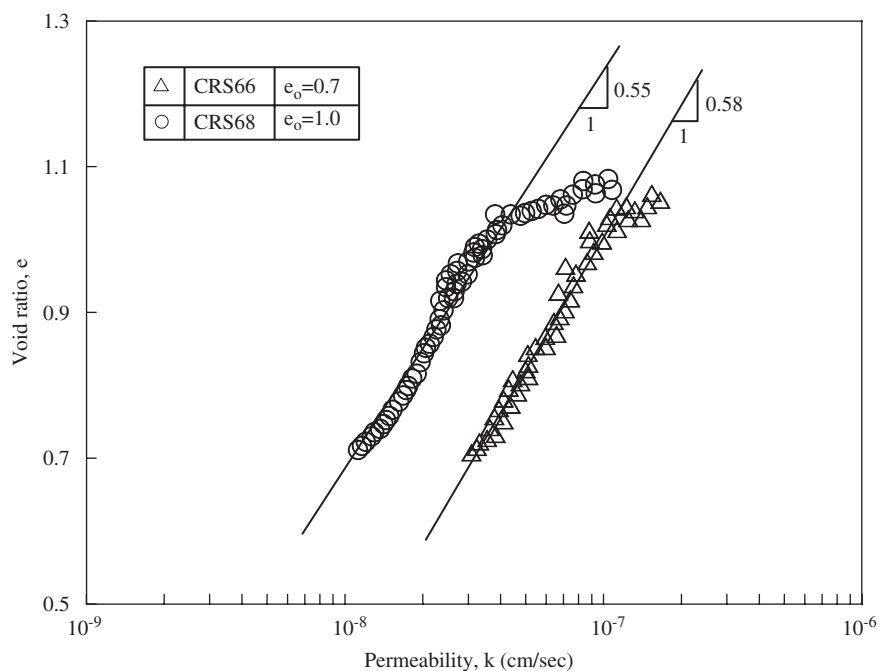


Figure 22. Relationship between coefficient of permeability and void ratio for Taipei Clay (after Chin & Liu 1997).

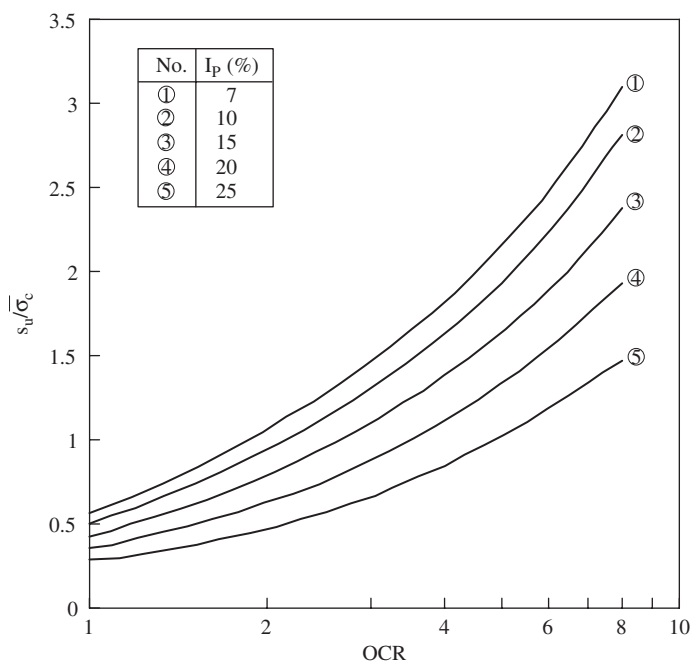


Figure 23. Relationship between undrained shear strength and OCR for Taipei Clay tested under CIUC (after Chin et al. 1989).

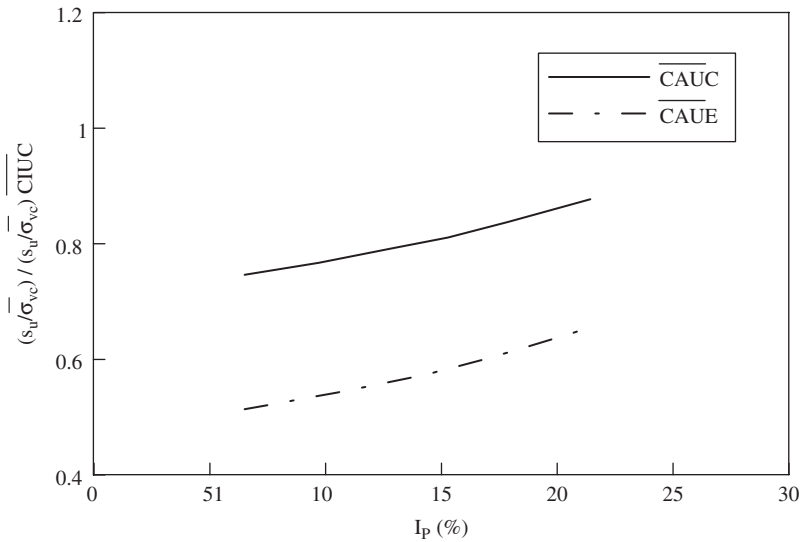


Figure 24. Effect of anisotropy and shearing modes on undrained shear strength of Taipei Clay (after Chin et al. 1989).

Table 9. Results of triaxial compression and extension tests of K1 clays (after Germaine 1992).

Depth (m)	Test no.	w_n (%)	e_o	Rate of consolidation (%/hr)	Result of consolidation					Note
					σ'_{vm} (kPa)	$(K_o)_{NC}$	σ'_{vc} (kPa)	OCR	$(K_o)_{OC}$	
19.77–19.95	TRI148	36	1.01	0.08	640	0.52	640	1	–	$\overline{CK_0UC}$
20.78–20.90	TRI158	39	1.08	0.08	700	0.55	700	1	–	$\overline{CK_0UE}$
18.85–18.95	TRI159	38	1.08	0.07	690	0.56	339	2.03	0.78	$\overline{CK_0UC}$
19.59–19.69	TRI160	37	1.05	0.08	700	0.52	348	2	0.69	$\overline{CK_0UE}$
20.64–20.74	TRI193	37	1.02	0.11	700	0.52	174	4.04	0.93	$\overline{CK_0UE}$
20.49–20.59	TRI194	40	1.12	0.09	670	0.56	171	3.92	1.07	$\overline{CK_0UC}$

Result of shearing											
Depth (m)	Test no.	Rate of shear (%/hr)	Maximum axial stress					Maximum obliquity			
			ϵ_a (%)	q/σ'_{vc}	p'/σ'_{vc}	ϕ' (deg)	μ	ϵ_a (%)	q/σ'_{vc}	p'/σ'_{vc}	ϕ' (deg)
19.77–19.95	TRI148	0.4	0.4	0.29	0.711	24.1	1	10	0.252	0.493	30.7
20.78–20.90	TRI158	–0.49	–13.3	–0.192	0.351	33.2	1	–13	–0.192	0.35	33.3
18.85–18.95	TRI159	0.16	1.7	0.51	1.127	26.9	0.2	6.5	0.484	1.012	28.3
19.59–19.69	TRI160	–0.32	–13.7	–0.329	0.57	35.2	0.79	–12.6	–0.326	0.561	35.4
20.64–20.74	TRI193	–0.5	–12.4	–0.633	1.607	36.4	0.41	–11.9	–0.631	1.056	36.7
20.49–20.59	TRI194	0.2	3.55	0.832	1.738	28.6	0.08	3.55	0.832	1.738	28.6

The relationship between K_o and OCR has been established using results obtained from these triaxial tests (Chin & Liu 1997). Figure 25 shows this relationship for the four overconsolidated samples. As can be seen from this figure and Table 9, the value of K_o at NC state ranges from 0.52–0.58, and it increases with the increasing OCR. The results from these four tests fall in a small range. To describe the K_o – OCR relationship, an empirical correlation based on effective stress friction angle was suggested by Mayne & Kulhawy (1982). When $\phi' = 28^\circ$ is used in this empirical correlation, it can be seen that the estimated results agree closely with the test results both in trend and in value.

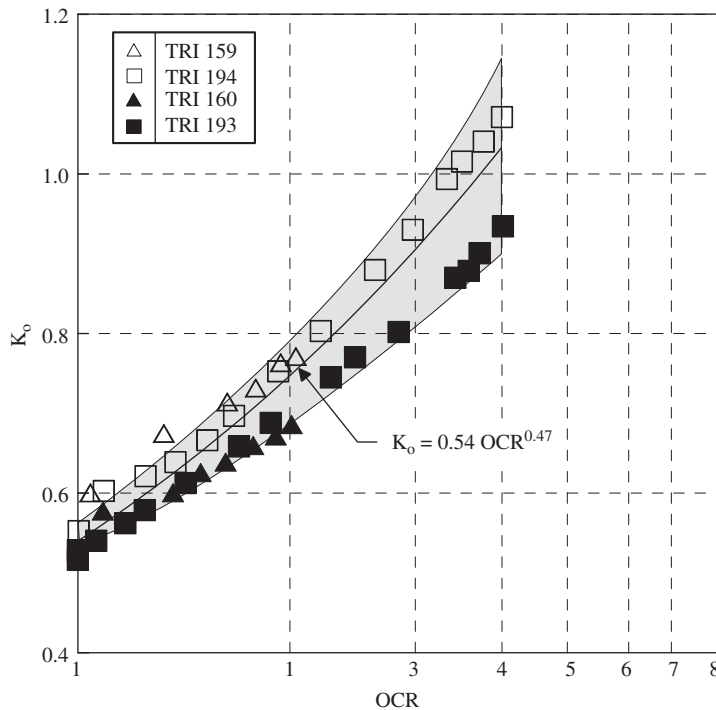


Figure 25. K_o -OCR relationship for Taipei Clay (after Chin & Liu 1997).

The results of this series of tests are presented in Figure 26 as the stress-strain curves and in Figure 27 as the stress paths. Under normally consolidation conditions, the values of undrained shear strength ratio (s_u/σ'_{vc}) are 0.29 and 0.19 in compression and extension, respectively. The friction angle in compression, $\phi_{TC}' = 28^\circ$ is well defined from effective stress measurements at axial strain levels, $\epsilon_a \approx 10\%$. The equivalent friction angle in extension, ϕ_{TE}' , is more difficult to estimate due to specimen necking. The selected friction angle, $\phi_{TE}' = 33^\circ$ was estimated at $\epsilon_a \approx 10\%$ (Whittle et al. 1993).

The normalized undrained secant modulus (E_u/σ'_{vc}) are plotted versus the axial strain in Figure 28. The degradation of modulus with increasing axial strain is apparent. For CK_oUC test results, it is found that the values of E_u/σ'_{vc} increase with increasing OCR at the same level of ϵ_a . For CK_oUE tests, the OCR influence is not obvious, which suggests that the effect of previous overconsolidation is greater on vertical undrained modulus than on the horizontal one. Kung (2003) studied behavior of Taipei Clay at small strain. Deformation modulus of samples taken from K1 Zone was measured using bender elements in triaxial tests. It was found that the results of E_u/σ'_{vc} obtained from the small strain tests are close to those shown in Figure 28 over the same axial strain range. When comparing the maximum shear modulus obtained in laboratory and at site, it was found that the value measured by the bender element is slightly lower than those measured by in-situ down-hole and cross-hole tests (Kung 2003).

5.3.3 Direct simple shear tests

In addition, a single undrained direct simple shear test (CK_oUDSS) was also performed on a normally consolidated specimen of the clay. The stress-strain curve of this test is shown in Figure 29, and the stress path of this test is given in Figure 30. At a shear strain of about 10.2%, the maximum shear stress is reached. The corresponding undrained shear strength ratio (τ_h/σ'_{vc}) is about 0.23.

Another set of direct simple shear tests were conducted by Liu (1999). In this study, tests were performed at various overconsolidation ratios. The stress-strain relationship for these DSS tests are shown in Figure 31, while the corresponding stress paths are given in Figure 32. For normally consolidated sample, the MIT test result is very close to that obtained in this study. As can be seen, the undrained shear strength of the clay is a function of OCR. These test results are useful for establishing the SHANSEP equation, as discussed later.

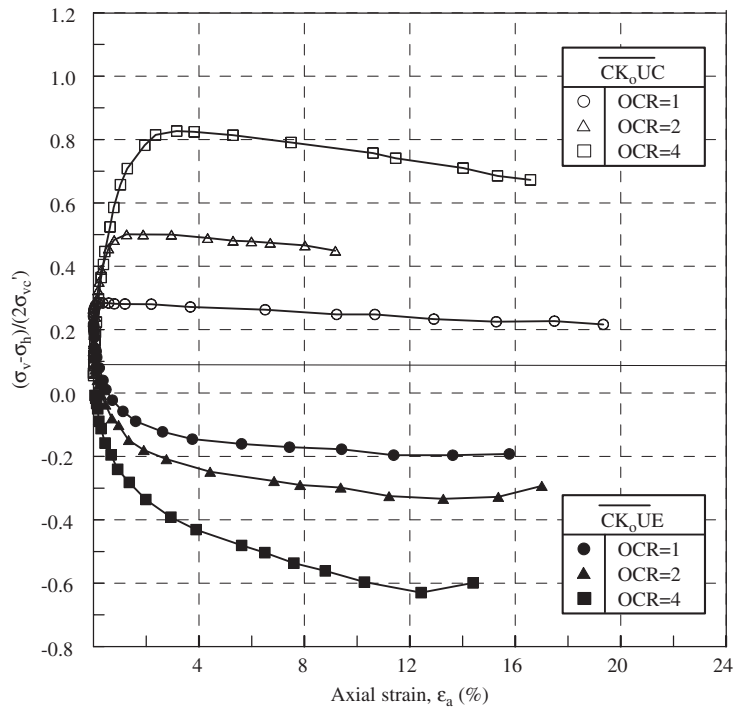


Figure 26. Stress-strain curves for six triaxial tests of Taipei Clay (after Chin & Liu 1997).

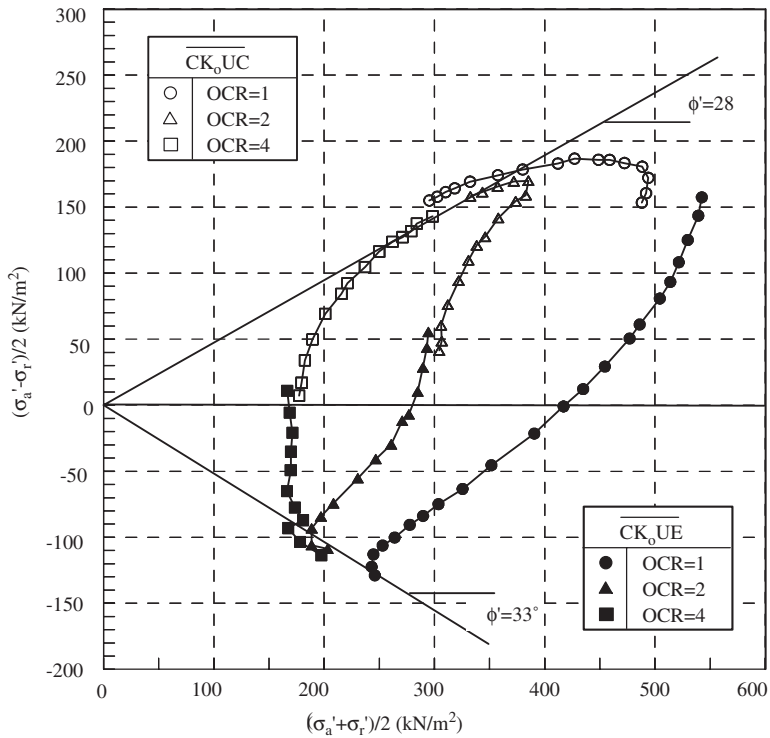


Figure 27. Stress paths for six triaxial tests of Taipei Clay (after Chin & Liu 1997).

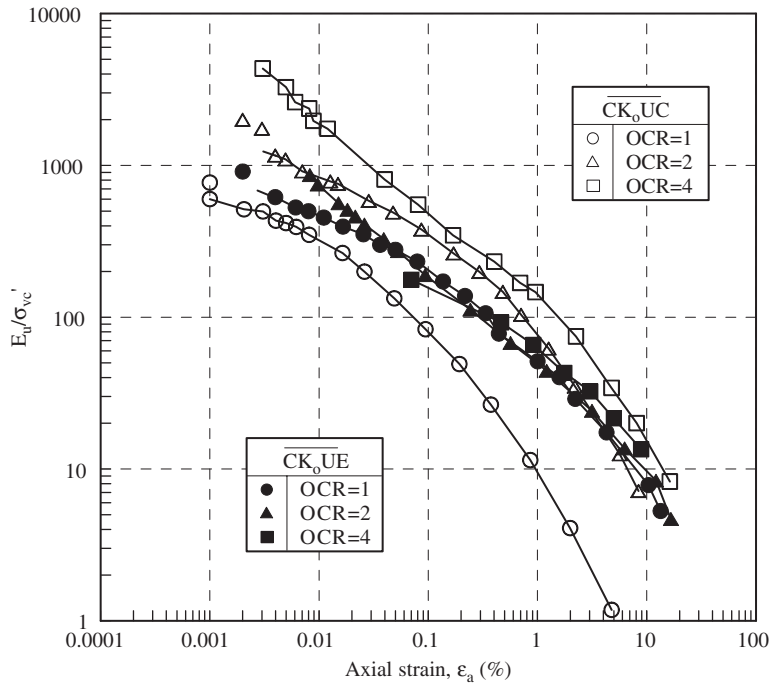


Figure 28. Normalized undrained modulus degradation for six triaxial tests of Taipei Clay (after Chin & Liu 1997).

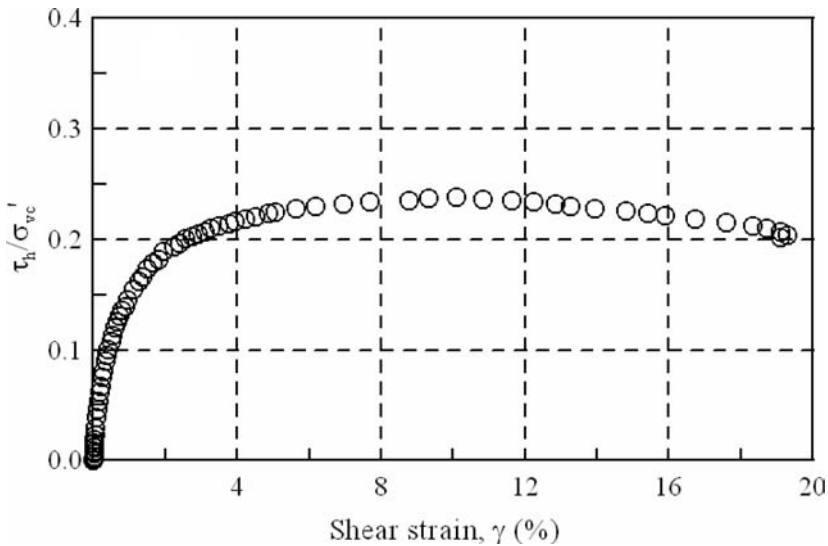


Figure 29. Stress-strain curve of Taipei Clay under CKoUDSS tested at MIT (after Germaine 1992).

5.3.4 SHANSEP strengths

For normalized strength behavior of Taipei Clay, undrained shear test results from samples obtained at various locations were studied. At K1 site, samples were taken at sufficient depth to be below the highly over-consolidated upper zone. Both compression and extension tests were performed (CK₀UC and CK₀UE tests), and results were given in the previous section. Samples of Layer IV soils from various depths and locations also were tested, and they were normally consolidated under K₀ conditions in triaxial cells (CK₀UC tests).

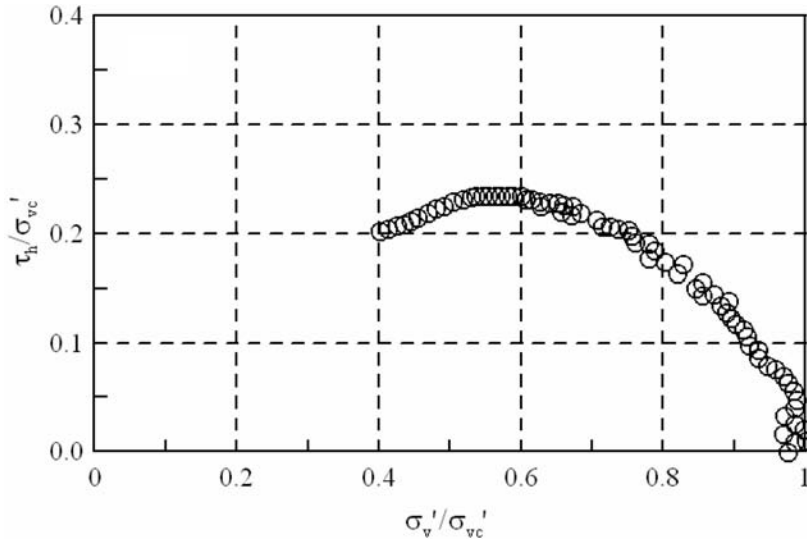


Figure 30. Stress path for Taipei Clay under CKoUDSS tested at MIT (after Germaine 1992).

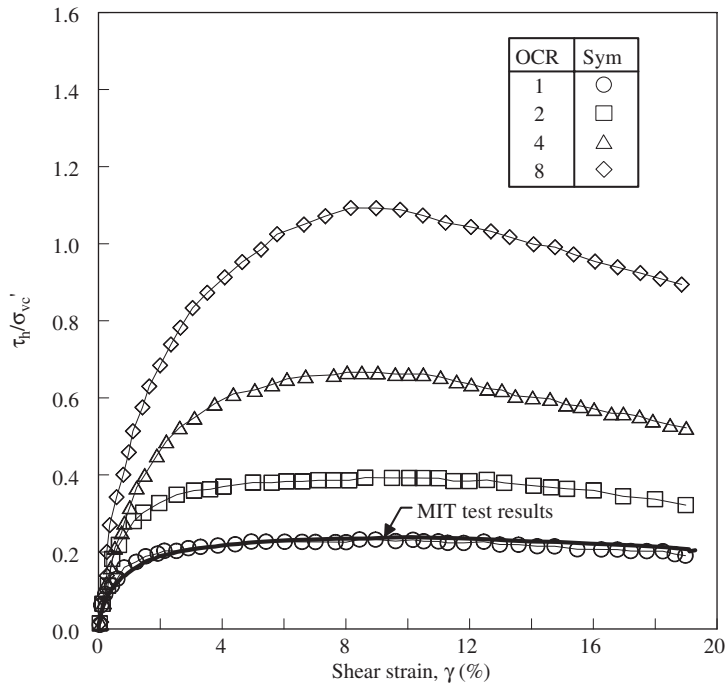


Figure 31. DSS stress-strain curve for Taipei Clay (adapted from Liu 1999).

Results of compression tests were compiled in Figure 33, where the undrained shear strength ratio is plotted versus OCR. The majority of these strength test data lie in a relatively narrow band regardless of location. The relationship between the compression strength and OCR_v can be established, and the mean relationship can be expressed by the equation given below:

$$s_u/\sigma_{vc}' = 0.32 OCR_v^{0.82} \quad (4)$$

It is noted that some normally consolidated samples exhibit high normalized strengths. This phenomenon can be attributed to dilation. For this type of effect, the SHANSEP approach does not apply, and these

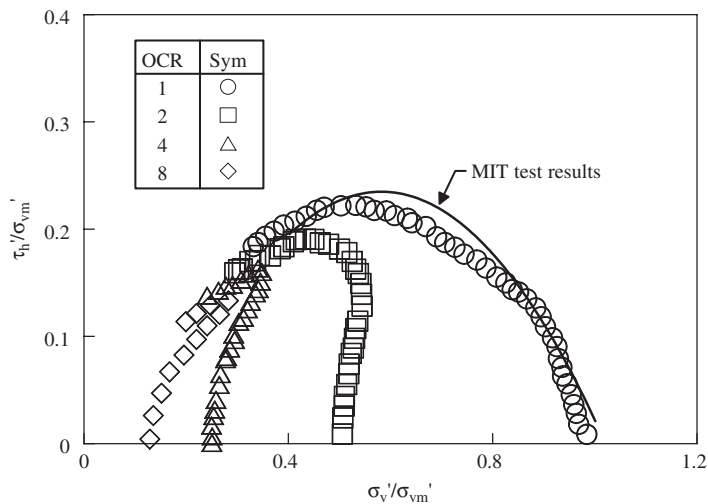


Figure 32. DSS stress paths for Taipei Clay (adapted from Liu 1999).

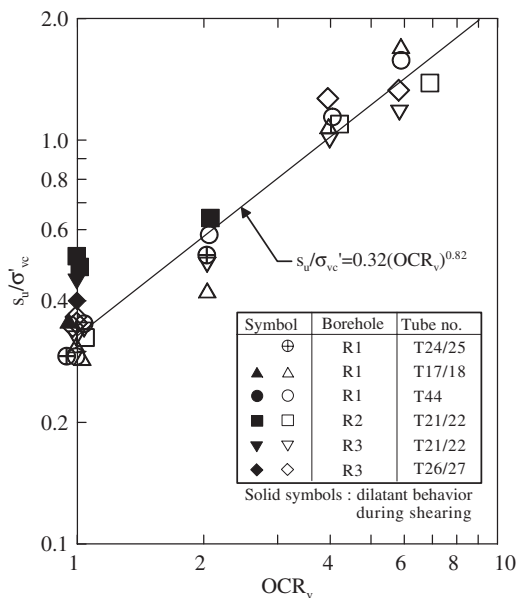


Figure 33. Undrained compression strength ratio versus OCR from CK_oUC tests (after Chin et al. 1994).

data were excluded for interpretation of relevant normalized soil behavior. It should be noted that tests run at MAA Laboratory is about 8.9%/hr, which is about 17 to 55 times of that used at MIT. The pore water pressure response during undrained shear in compression is shown in Figure 34, and the behavior is typical for a wide range of clays except for those samples exhibit dilation.

For undrained extension, only two sets of tests were performed following the SHANSEP procedure, and the results are shown in Figure 35. Nevertheless, consistent relationships between undrained shear ratios and OCR still can be established. The correlations are shown in the figure and are given below for convenience:

$$s_u'/\sigma'_{vc} = 0.19 \text{OCR}_v^{0.82} \quad (5)$$

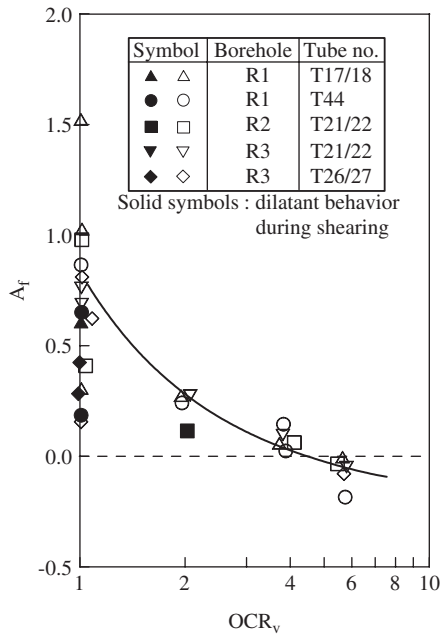


Figure 34. Porewater pressure response during shearing from CKoUC tests (after Chin et al. 1994).

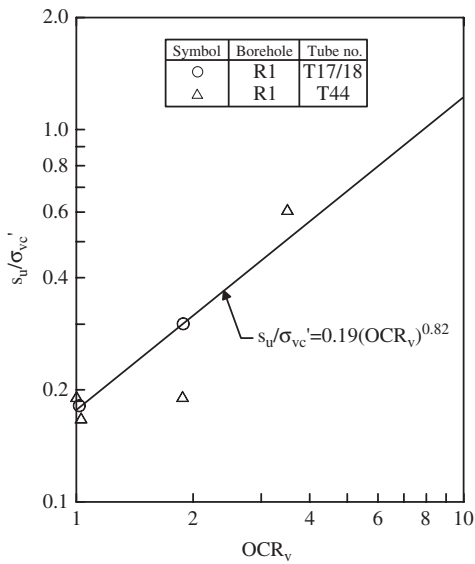


Figure 35. Undrained compression strength ratio versus OCR from CKoUE tests (after Chin et al. 1994).

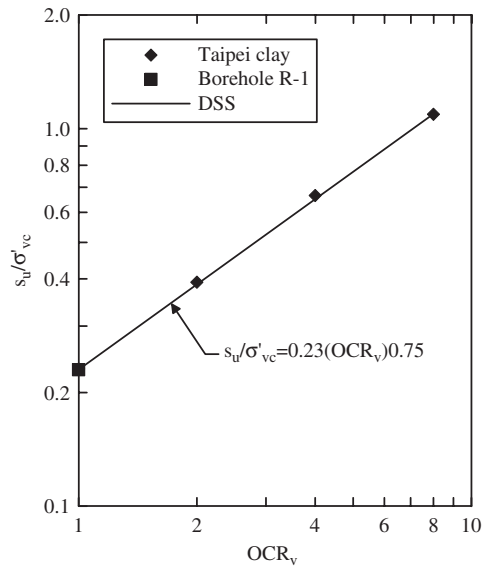


Figure 36. Undrained compression strength ratio versus OCR from DSS tests (after Chin et al. 1994).

For DSS tests, results shown in a previous section were studied. Figure 36 shows the plot of undrained shear strength ratio versus OCR. The normalized behavior can be established, and the correlation can be described as follow:

$$s_u/\sigma'_{vc} = 0.23 OCR_v^{0.75} \quad (6)$$

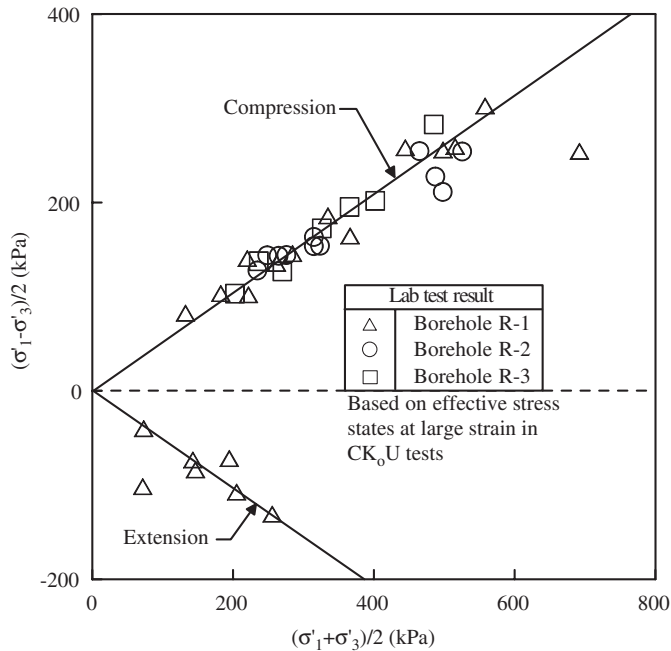


Figure 37. Effective stress strength envelopes of Taipei Clay (after Chin et al. 1994).

5.3.5 Effective stress friction angle

The effective stress states at the end of consolidated undrained tests also provide information on the effective stress friction angle of soils. Figure 37 compiles results of all K_0 consolidated undrained triaxial tests both in compression and in extension. As can be seen, little scatter exists and the average effective stress friction angle for all cohesive Sungshan deposits is about 32 degrees both in compression and in extension.

5.3.6 Comparisons of results from different tests

To investigate the effect of different reconsolidation procedures on the undrained shear strength, tests were carried out using samples taken from Boreholes R-1, R-2, and R-3. Six CK_0 UC tests were performed on these samples using recompression (reconsolidated to the in-situ effective stress state) procedures (Bjerrum 1973). Results obtained from these tests were compared with the strengths predicted using the SHANSEP approach in Figure 38, and good agreement between these two types of approaches are observed (Chin et al. 1994).

Undrained shear strengths measured using saturated unconsolidated undrained (SUU) shear test and unsaturated unconsolidated undrained (UUU) shear tests on samples from K1 and K1/T2 zones were compared with results derived from the normalized strength relationships and those tested using recompression procedures in Figure 39. As can be seen, the UUU and SUU test results are consistently lower, and the results of SUU tests are particularly poor. The reason for this difference is simply that the effective stresses under UUU tests are unknown because pore water pressures are not measured, and consolidation stress before SUU test shearing is only 5 kPa at MAA Laboratory (i.e. sample consolidated is under a confining stress 5 kPa larger than the back pressure). It is obvious that the actual effective stresses in UUU and SUU tests are significantly lower than those exist at site.

6 IN-SITU TESTS

In Taipei, one of the most widely used field tests in this region for soil properties characterization is the standard penetration test (SPT). Other commonly used in-situ test methods include field vane shear tests, cone penetration tests, pressuremeter tests. Geophysics probing techniques such as the electrical resistivity method and downhole velocity logging method also have been used in many projects. Most in-situ tests summarized in this section were conducted in the area adjacent to the Taipei City Hall Station

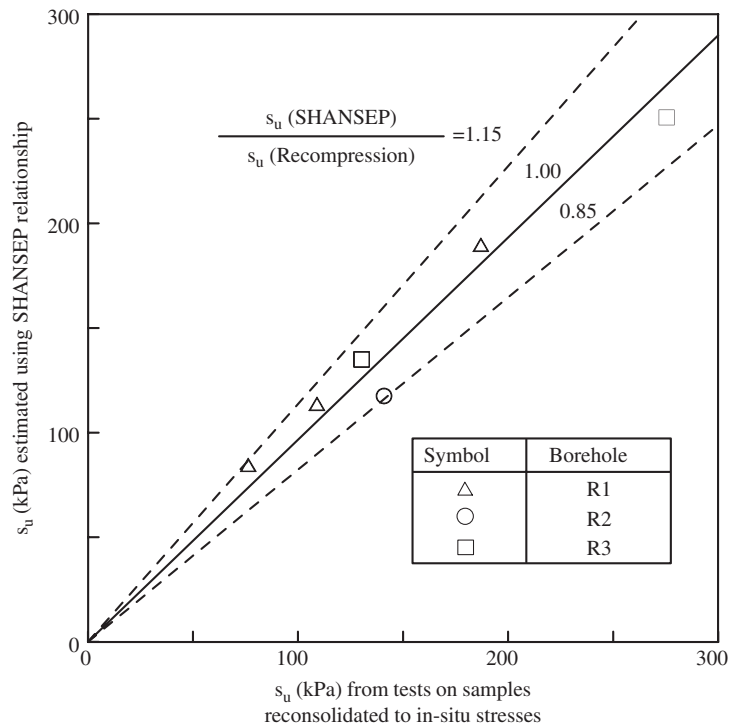


Figure 38. Comparison of s_u between recompression and SHANSEP type of tests (after Chin et al. 1994).

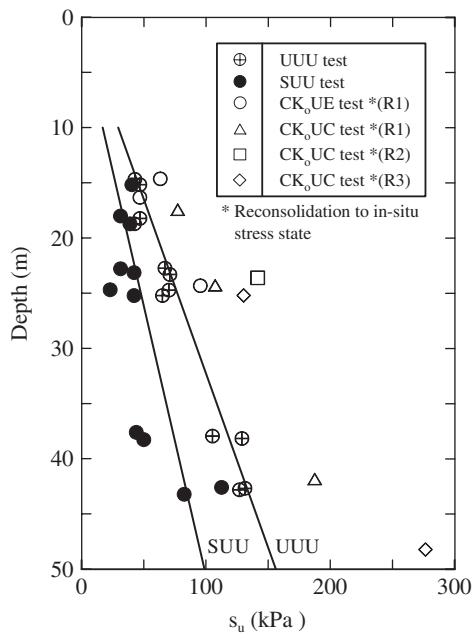


Figure 39. Comparison of s_u from various test types (after Chin et al. 1994).

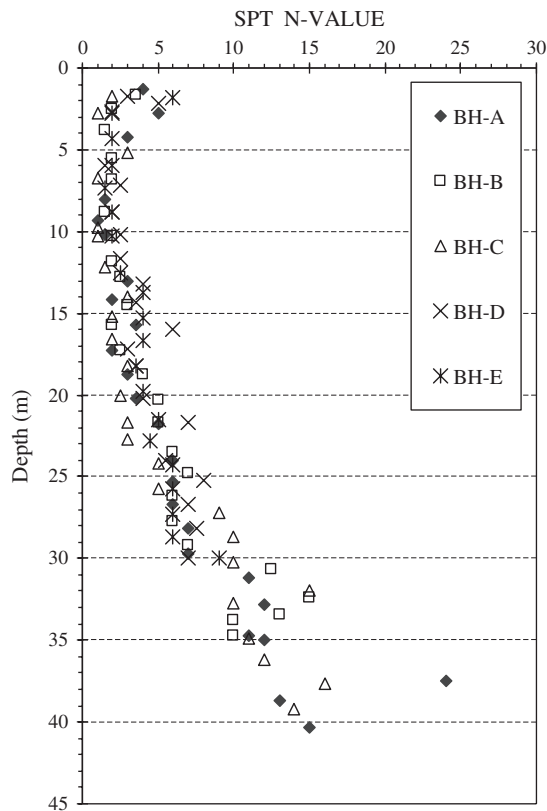


Figure 40. Typical SPT-N profile of K1 zone in Taipei.

of TRTS, which is located in K1 Zone. The typical profile of the site is represented by the profile of Borehole R-1 shown in Figures 12 and 13.

6.1 Standard penetration test

Standard penetration tests are used very extensively in a routine site exploration program. It provides data for design with the use of empirical correlations. In addition, disturbed samples can be obtained for physical property tests in laboratory. In general, a great amount of physical property data often can be obtained in a typical site exploration project. In Taiwan, SPT is carried out according to ASTM 1586. However, it should be noted that brass liners are always used together with the split spoon barrels. After samples were taken, they were sealed right the way and sent for subsequent laboratory tests. It is noted that although many of these data may not be useful (e.g. water content of cohesionless soil obtained from a split spoon sample), a lot of data on physical properties have been accumulated because of this practice. They have become very useful once a systematic framework has been set up for geotechnical zoning works. Statistics of physical properties shown in Tables 4 to 8 were able to be compiled as a result of significant amounts of data collected. A typical SPT-N value profile for the K1 soil is shown in Figure 40.

6.2 Field vane shear test

Field vane (FV) shear tests were used widely to obtain the in-situ undrained shear strength for Taipei cohesive deposits. For Taipei Clay, tests conducted in several locations in close proximity within the K1 Zone were available, and they can be used to portrait the s_u profile of the clay, as shown in Figure 41. The undisturbed and remolded strength values are shown in Figure 41a, where the undrained strength corrected using Bjerrum's factors, μ , (Bjerrum 1972) also is shown in the figure. As can be seen, no

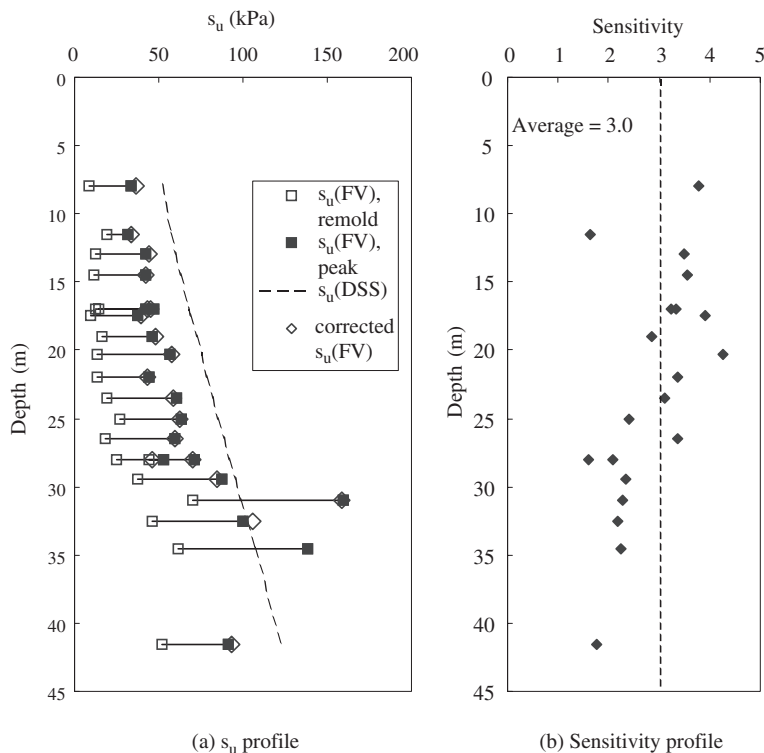


Figure 41. Field vane shear s_u profile for Taipei Clay.

appreciable difference can be depicted between the corrected and uncorrected peak FV strength for this clay. For comparison, the s_u profile based on the direct simple shear SHANSEP equation (Equation 6) also is plotted in Figure 41a. It can be seen that the trend of the $s_u(FV)$ profile is quite consistent with the $s_u(DSS)$ profile, but the value of $s_u(FV)$ is somewhat smaller than that of $s_u(DSS)$. On average, the $s_u(DSS)/\mu s_u(FV)$ is about 1.5 for normally consolidated to slightly overconsolidated Layer IV Taipei Clay. Using the state profile for K1 soil shown in Figure 10, values of $s_u(FV)/\sigma'_p$ can be calculated, and the computed value is about 0.13 on average. This is much smaller than that suggested by Larsson (1980), where it is suggested that the $s_u(FV)/\sigma'_p$ ratio be likely about 0.23 ± 0.04 .

The sensitivity of the clay is shown in Figure 41b, and the value is about 3.0 on average. In another site investigation report (MAA 1983), results of unconfined compression strength tests on K1 Clay showed that the sensitivity ranges from 4 to 8 with an average of 5.9. In that study (MAA 1983), it also was found that the FV sensitivity is always lower (smaller than 4.0).

6.3 Cone/Piezcone penetration test

The electric cone penetration test was generally conducted in accordance with ASTM standard D3441. During the development of the initial network of the TRTS, the electric cone penetration tests were carried out extensively (Wong et al. 1993). Cones equipped with pore pressure transducer to measure the excess pore pressure developed while conducting the test are known as piezocone, and the enhanced procedure is known as the piezocone test. In Taipei, the piezocone test is becoming increasingly popular for practicing engineers in characterizing soils in recent years. General applications of this test include detecting soil variability, classifying soil, developing OCR profile, finding undrained shear strength, estimating consolidation coefficient and permeability, and more. Typical result of a piezocone test on K1 clay is illustrated by Figure 42 based on a set of data retrieved from a site near the Sungshan Airport in which the cone tip resistance, Q_t , local sleeve friction, F_s , friction ratio in terms of F_s/Q_t and pore pressure are plotted versus depth.

One of the methods for determining undrained shear strength is presented herein as an example to demonstrate the application of the CPT. In regional practice, the s_u of a clayey soil can be determined

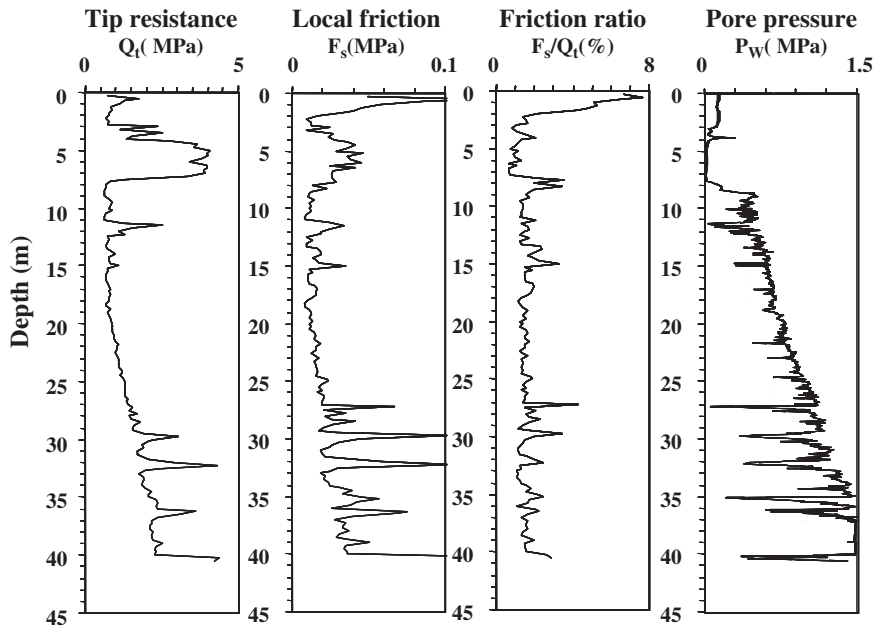


Figure 42. Typical CPT results for Taipei Clay.

by substituting the measured cone tip resistance, q_c , the total overburden stress, σ_{vo} , and the cone factor N_k at given depths into the following equation:

$$s_u = (q_c - \sigma_{vo}) / N_k \quad (7)$$

The value of N_k is a result of calibration based on available CPT data and s_u value measured from DSS tests conducted at given depths. For normally consolidated and slightly overconsolidated K1 clay, the N_k value is approximately 8.5 on average.

A variety of soil parameters can be determined from the CPT. Using this method, the ground variability can be detected more effectively compared to most of the other approaches because the data are retrieved continuously during the test. However, as being mentioned, the early CPT devices tended to have lower than expected system rigidity which often resulted in unwanted errors. In addition, it has been found that the deairing process for the porous material in the device was sometimes not as good as expected. It is probably due to operator's negligence or unfamiliarity with the test procedure. As reported by Wong et al. (1993), this can cause lag of the piezocone response to the pore pressure change and result in ineffectiveness in detecting thin layers. Figure 43 shows a case history in which a typical response of the piezocone to the pore pressure measured by poorly saturated device was shown in part (a) and the response measured by a better saturated device was given in part (b). As can be seen, the measured response in Figure 43a is almost meaningless whereas apparent indication of change in ground properties and pore pressures can be identified in Figure 43b.

6.4 Pressuremeter test

The pressuremeter test often is called the lateral load test (LLT) due to local preference. Commercially available testing equipment include Menard pressuremeter from France, OYO's K-value tester, LLT and KKT from Japan. One of the commonly used apparatus in local practice is the LLT device made by OYO Corporation of Japan. This test method is generally performed to evaluate the deformation parameters of the subsoils such as the coefficient of horizontal subgrade reaction, coefficient of earth pressure at rest, shear modulus and elastic modulus. For K1 clay, a typical LLT result is shown in Figure 44. The LLT results also can be used to estimate the undrained shear strength using the following equation:

$$s_u \text{ (LLT)} = (p_L - p_o) / N_q \quad (8)$$

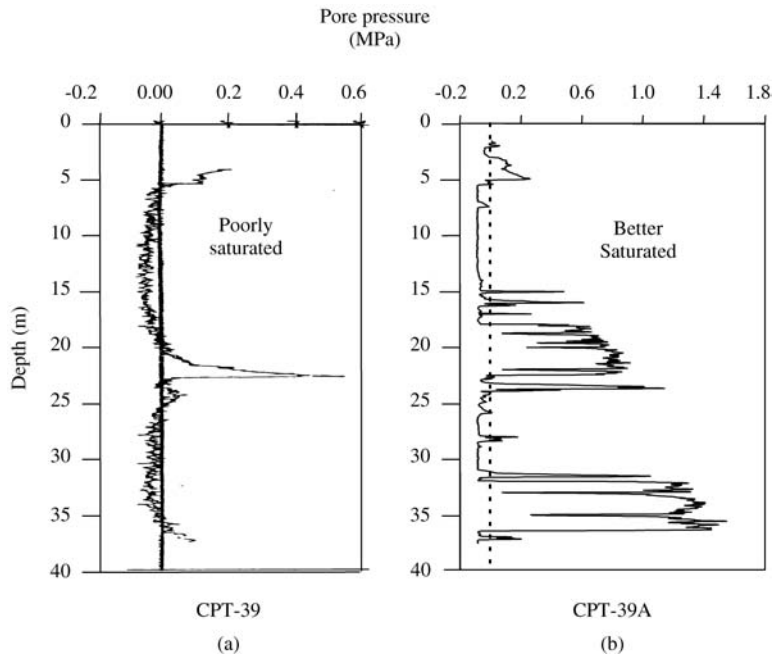


Figure 43. Effects of saturation of filter tip on CPT results.

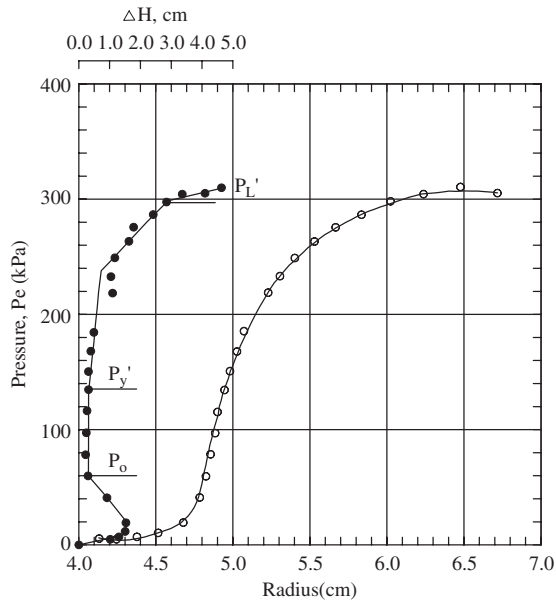


Figure 44. Typical LLT results of Taipei Clay.

in which p_L = LLT limit stress, p_o = LLT total horizontal stress, and N_q = LLT factor. Values of N_q for Taipei Clay have been evaluated with reference to the s_u measured from DSS tests. It was found that for Taipei Clay the N_q value is about 4.4 based on seven sets of available data. Modulus and K_o can also be derived from LLT. It has to be noted that results for K1 clay are much smaller than the typical values reported by others (e.g. Kulhawy & Mayne 1990). The reason can probably be attributed to the amount of significant disturbance.

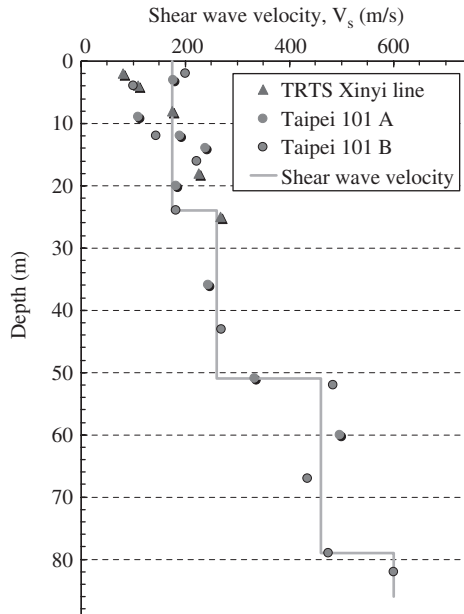


Figure 45. Shear wave velocity profile for Taipei Clay.

6.5 Shear wave velocity test

The most common seismic methods in the context of geotechnical engineering used in Taipei region perhaps is the down-hole velocity logging. This method consists of generating shock waves in the ground, allowing them to propagate through soil, then using sensors in a predrilled borehole to measure the wave velocity as they arrive at specified depths. Appropriate velocity-depth functions and direct correlation between seismic reflections and depth can be obtained from this method. Figure 45 shows a typical shear wave velocity profile in the K1 Zone. This plot was established based on test results conducted at the construction sites of Taipei 101 and TRTS Xinyi Line.

6.6 Electrical resistivity method

Electrical resistivity method is one of the electrical prospecting. This method has been used routinely in TRTS projects for the purpose of obtaining parameters for electricity system design. The objective of this method is to identify the structure of the resistivity layers based on the apparent ground resistivity. A typical plot of vertical electrical resistivity sounding and resistivity log is illustrated herein by one of the tests conducted at T2 Zone as shown in Figure 46. Although this is one of the approaches that often used to investigate ground profile composition, it is very sensitive to environmental interference and operator's craftsmanship.

7 ENGINEERING PROBLEMS

Due to the high-density urban development in past decades and the decreasing availability of construction space, more and more underground construction projects such as cut-and-cover excavations and tunneling have to extend their depths to Layer IV or deeper layers of Sungshan Formation in recent years. This brought great concerns to the potential of differential settlement, which is often a result of underground construction activities and can be detrimental to the structure of the buildings being influenced. Impact of new construction project to adjacent structures has to be taken into consideration during design and construction.

With the high sensitivity and the closer to or higher than the liquid limit natural water content of the Layer IV clay, construction activities often result in shear strength reduction as the ground is subject to

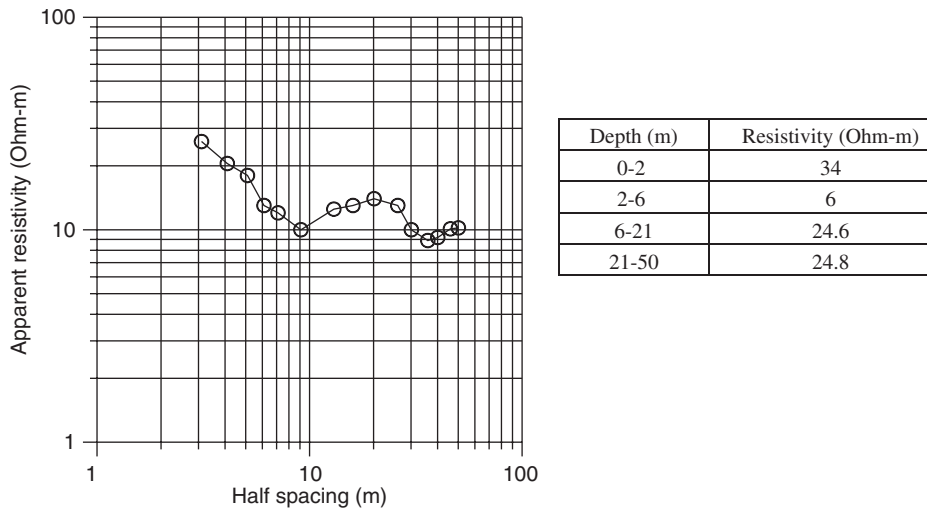


Figure 46. Typical results of vertical electrical resistivity sounding curve and resistivity log for Taipei Clay.

disturbance. For cut-and-cover construction, to ensure the stability of soil below the excavation surface and the safety of the infrastructures nearby, the retaining structures need to penetrate into a depth significantly greater than the final excavation depth. Because the mobilized passive earth pressure is usually lower than that expected, large lateral displacement of the retaining walls will occur and associate with significant amount of ground settlement. To restrain this lateral displacement, prevent basal heave and mitigate the potential of differential settlement on the ground outside the excavation zone, high stiffness and elongated retaining walls, prestressed bracing struts, barrette, cross panel, and ground improvement measures such as high pressure jet grouting for soils outside the excavation zone and below the excavation level usually were taken.

Another example demonstrated herein is tunneling. The ground settlement over tunnels due to tunneling was primarily the results of the following phased construction activities including (a) shield machine face advancement, over-cutting and shield advancing, (b) the closure of tail void between the shield machine and surrounding soil, and (c) consolidation. Hwang, et al. (1995) combined the phases (a) and (b) as “ground loss settlement” and singled out consolidation settlement from others. For soft clays, the consolidation settlement may be a predominant source of settlements and may drag on for days, if not months. Similar to deep excavation aforementioned, ground settlement resulted from tunneling can be detrimental if the ground behavior was not appropriately evaluated beforehand.

Prior to the initiating of the TRTS project, experience consisting of empiricism and precedent seemed to play a lopsided role in general geotechnical engineering practice in the region of Taipei. With increasing demands to tackle project under complex construction conditions and resolve the engineering problems that engineers must face, in-depth understanding of soil behavior became a critical issue. Nowadays the empirical or semi-empirical procedures still plays an important role in geotechnical engineering practice, other approaches such as physical modeling, analytical modeling, or numerical modeling based design gradually gain their popularity and importance. It probably can be attributed to the state-of-art mechanical and electrical field and laboratory testing equipment and the fast developed computer technologies which in turn make the once seemingly difficult or impossible task an easy one now.

One of the most powerful tools for solving today's engineering problems is the joint application of realistic soil models and advanced numerical modeling in design practice. An example given herein is related to a soil-structure interaction problem. Two bored tunnels planned to be excavated by shield machine will go across an existing cut-and-cover twin box tunnel from underneath. One of the twin box tunnel consists of the Taiwan High Speed Rail tracks and the Taiwan Railway tracks are in another. The bored tunnels are part of the new Sungshan Line of the TRTS near the Taipei Main Station (Chao et al. 2005). As far as the engineer is concerned, interruption of the regular operation of the High Speed Rail by any of the adverse construction effects is the risk that can not be affordable. There was no precedent case available and thus empirical or semi-empirical design method was not an option. The 2-D simulation approach was inappropriate herein because the potential influence of the construction activities to the box tunnels was 3-D in essence as can be seen in Figure 47. Thus, Class-A 3-D evaluations were conducted

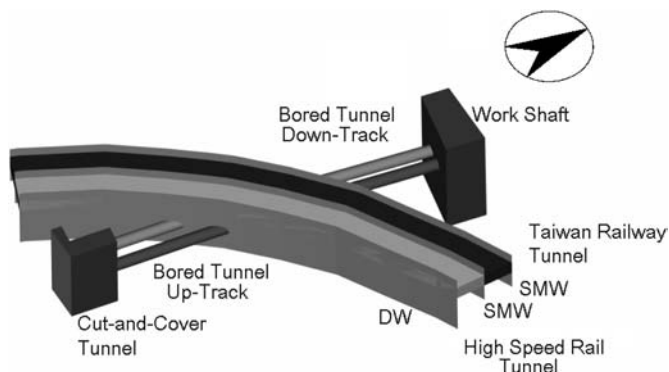


Figure 47. Proposed construction of new Sungshan Line of the TRTS (after Chao et al. 2005).

Table 10. Evaluated MIT-S1 parameters for Taipei Clay (Pestana & Whittle 1999, Hsieh 2006).

Test Type	Parameter	Physical contribution/meaning	Boston blue clay	Taipei Clay	Lower cromer till
1-D Consolidation (Oedometer CRS ,K ₀ -triaxial)	ρ_c	Compressibility of limiting compression Curve (LCC)	0.178	0.16	0.167
	D	Non-linear volumetric	0.04	0.01	0.04
	r	Swelling behavior	0.85	0.8	0.8
	h	Irrecoverable plastic strain	6	15	6
K ₀ -triaxial	K _{0nc}	K ₀ for NC clay	0.49	0.52	0.50
K ₀ -triaxial OCR = 5~10	μ'_o	Poisson's ratio at stress reversal controlling 2G _{max} /K _{max}	0.24	0.21	0.225
	ω	Non-linear Poisson's ratio. Stress path in 1-D unloading	1	1	0.6
Undrained triaxial shear tests:	ϕ'_{cs}	Critical state friction angle in triaxial compression	33.5°	30°	30.0°
CK ₀ UC (OCR=1)	ϕ'_m	Geometry of bounding surface	46°	35°	42.0°
CK ₀ UE (OCR=1)	m	Stress path of undrained CK ₀ UTC/TE tests	0.8	1.4	0.65
CK ₀ UC (OCR=2)	ω_s	Non-linear at small strains in undrained shear	8	12	8.0
Resonant column or shear wave velocity	C _b	Small strain stiffness at load reversal	450	450	450
CK ₀ UC	ψ	Rate of evolution of anisotropy	15	8	20
CK ₀ UE		(rotation of bounding surface)			

in the design phase to obtain insight into various soil-structure interaction behaviors associated with the complicated nature.

To deal with complexity, an advanced numerical modeling using MIT-E3 model was established for Taipei Clay at the same time the initial network of the TRTS was implemented. Without providing field monitored information prior to the modeling, satisfactory prediction results were obtained for the deep excavation of the Convention Center of Taipei World Trade Center (Whittle et al. 1993). More recently, an updated and unified constitutive model for both sand and clay, namely MIT-S1, was developed by Pestana and Whittle (1999). Significant amount of endeavors have been put into it for evaluating the parameters and assessing the effectiveness of this model for Taipei Clay.

Table 10 tabulated the results of the parametric evaluation for Taipei Clay. For comparison, parameters for Boston Blue Clay and Lower Cromer Till also are provided. Table 11 summarized the results of

Table 11. Comparison of measured and predicted undrained shear strength ratios for Taipei Clay.

	MIT-S1	Measured	MIT-S1	Measured	MIT-S1	Measured
OCR	1	1	2	2	4	4
K_o	0.52–0.55	0.52–0.55	0.7–0.77	0.7–0.77	0.9–1.06	0.9–1.06
$s_{uTC}/\sigma'_{vc} \text{ (1)}$	0.30	0.29	0.54	0.51	0.86	0.83
$s_{uTE}/\sigma'_{vc} \text{ (1)}$	0.18	0.19	0.32	0.33	0.60	0.63
s_{uPSA}/σ'_{vc}	0.32	—	0.55	—	0.90	—
s_{uPSP}/σ'_{vc}	0.24	—	0.42	—	0.73	—
$s_{uDSS}/\sigma'_{vc} \text{ (1)}$	0.2	0.23	0.39	—	0.65	—
$s_{uDSS}/\sigma'_{vc} \text{ (2)}$		0.23		0.39		0.67

Note: measured values for (1) from Whittle et al. (1993); (2) from Liu (1999).

comparison between measured and predicted values of various modes of undrained shear strengths at a variety of OCR values based on the evaluated parameters. As can be seen, results of the elementary level evaluation showed that the MIT-S1 model was capable of providing excellent predictions on the undrained shear strength ratios for Taipei Clay. Thus, it is very encouraging to carry on with further investigation to evaluate the prototype engineering problems. Nevertheless, engineers have to bear in mind that the MIT-S1 model is a rate independent model. Further studies on rate dependent effect for Taipei Clay are required.

8 SUMMARY AND CONCLUSIONS

Taipei City is located in the northern part of Taiwan and occupies more than half of the Taipei Basin. More and more building and infrastructure developments with increasing scale and significance are built. As a result, extensive investigations have been conducted on the soil deposits of Taipei, especially the cohesive deposits. This paper presents a comprehensive overview of behaviors and properties of Taipei Clay.

The Taipei Basin was formed as a result of reverse and normal faulting in northern Taiwan. The main soil deposit of the Basin is Sungshan Formation. The late Pleistocene to Holocene Sungshan Formation in the Taipei Basin was deposited under the fluvial, lacustrine, estuary and brackish water bay environments. The maximum thickness of Sungshan Formation is about 120 m. The greatest thickness of Sungshan Formation is in the northwestern Wuku area, and it gradually thins toward the southeast and the northeast. Near the end of deposition of the formation, the brackish water gradually retreated from the basin. Since Taipei region subsided around 0.3–0.4 Ma and the Taipei Basin has been formed, the area has undergone a continuous sedimentation process, probably without recognizable erosion. However, due to excessive water pumping from the underlying Chingmei Formation, groundwater pressure of Sungshan Formation have been lower and ground subsidence was resulted. Even though the water pumping is now prohibited by the government, it is very important that the groundwater pressure be continuously monitored.

A major step toward systematic understandings of the Taipei soils was achieved by the first city-wide geotechnical mapping conducted and completed in 1986. The city was divided into six geotechnical zones based on a very scrupulous review of physical properties of soil samples and the depositional environment of three rivers flowing into the Basin. The results had great implications for subsequent researches as well as engineering practices. Results of zoning and the associated statistical characterization of Taipei Clay were presented. This geotechnical zoning work should be further extended to cover the whole Basin with the same level of scrutiny as it was developed in the mid-1980's.

Engineering characteristics of Taipei Clay have been investigated for decades. A more comprehensive understanding was achieved through many major construction projects in the late 1980's and early 1990's. The compressibility of Taipei Clay has been studied both in CRS and oedometer tests. The CR value for Taipei Clay obtained from the CRS test is about 0.18, which is very comparable to that of the Boston Blue Clay (CR = 0.17). Permeability characteristics of Taipei Clay has not been extensively studied. A lot of efforts have been put on the study of strengths of Taipei Clay. Comparisons have been made between undrained shear strength commonly used in local practice with more controlled tests. SHANSEP undrained shear strength ratios are established for CKoUTC, CKoUDSS, and CKoUE modes of shearing. These results are also comparable to those of Boston Blue Clay.

To study these engineering characteristics, most samples were taken from construction projects. Though high quality low disturbance samples were always required, not many investigations on disturbance effect have been conducted. A preliminary study reported by Hu et al. (2005), based on the

concept of “Specimen Quality Designation” (SQD), showed that reasonably good undisturbed samples can be obtained following procedures used in the Designated Task. Investigation on the disturbance effect should be further studied in the future. It is noted that many test results presented herein are in general accordance with ASTM or other standards, yet rate effect is very important and deserves more attention. In addition, resedimentation technique should also be considered in investigation of engineering characteristic of Taipei Clay.

Various in-situ tests have been used in Taipei. However, some results have not been rationally interpreted in practice. Local correlations are not always available. More systematic review would be needed for further applications of in-situ tests in Taipei. More thorough theoretical validation (e.g. use of the strain path method) or calibration chamber tests are needed to establish local correlations. Nevertheless, based on a first order calibration, it was found that VST, CPT, and LLT can be used to estimate undrained shear strength fairly consistently. Using s_u (DSS) as the reference, it is found that the cone factor N_k is about 8.5 and the LLT factor is about 4.4.

In this paper, model predictions on single element level have been discussed. It is very encouraging that the MIT-S1 model gives very reasonable predictions on undrained shear strength for various modes of shearing at various OCRs. For tackling very complicated construction projects, advanced numerical modeling may require further enhancement of the available constitutive models. Small strain behavior of soil is important for accurate modeling, and only limited studies have been done for Taipei Clay (e.g. Kung 2003). For comprehensive modeling, the rate dependent behavior is also important and should be modeled correctly. Liu (1999) has initiated the study of rate dependent behavior of Taipei Clay, but more efforts on this subject are needed in the future. It will be a very exciting and challenging task to model the whole spectrum of soil deposits of the Taipei Basin, from silty sand to silty clay, in a unified framework expressed in a single set of parameters.

9 ACKNOWLEDGEMENTS

First of all the Authors would like to thank Dr. Za-Chieh Moh. He initiated the systematic study of Taipei soils and promoted the geometrical zoning of the Taipei Basin. Dr. Moh encouraged and guided the Authors to pursue a better understanding of Taipei Clay. The Authors are very grateful to Dr. Jui-Tun Chou for his valuable input on the geology of Taipei. The Authors are also very grateful to Prof. Ming-Lang Lin for his advice on geology and its relationship with the engineering properties of Taipei soils. The Authors want to thank Dr. Chuan-Chi Liu for providing valuable information and test data of Taipei Clay. The Authors especially appreciate a long-time working relationship with Dr. Liu in the study of Taipei soils. Prof. Yo-Ming Hsieh is greatly appreciated for providing useful comments and suggestions on soil modeling and comparisons between Taipei Clay and Boston Blue Clay. The Authors also thank Prof. Liang-Chien Chen and Dr. Jiann-Yeou Rau of Center for Space and Remote Sensing Research, National Central University for providing satellite image of Taipei. Colleagues at MAA, Ms. Hsiao-Han Wu, Mr. Ching-Hui Huang, Mr. Hsiao-Chin Tseng, Mr. Wei-Jun Chung, Dr. Sung-Ju Chan, Dr. Jung-Feng Chang, Mr. Chung-Ren Chou, Mr. Cheng-Chen Wu, Mr. Pei-Lin Hou, and Dr. Tian Ho Seah, help prepare this manuscript. Their support and assistance are very much appreciated. Appreciation is also due to the Organizing Committee, Prof. Tan Thiam Soon and Prof. Phoon Kok Kwang, for inviting the Authors to present this paper.

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Characterization of alluvial deposits in Mekong Delta

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ABSTRACT: Soil investigations of soft alluvial soils in Mekong River Delta were carried out at two sites, Can Tho and Tan An, South Vietnam. The Can Tho site is beside the levee of Hau River, one of the major streams of Mekong with very large discharge rate and the Tan An site is beside Bam Co Tay River, a meandering river with relatively lower discharge rate. The differences of depositional environments create different types of soil profiles at the two sites, relatively low plastic soils with significant heterogeneity in the vertical direction at the Can Tho site and high plastic uniform clay at the Tan An site. In the investigation, soil samplings with Japanese type thin-walled tube sampler with fixed piston (JFP) and Shelby tube samplers were conducted. Consolidation and strength properties of the clays retrieved by the two samplers were studied. Various in-situ tests were also conducted at the site, such as piezocone penetration test, Marchetti's flat dilatometer test, a field vane test. Although carbon isotope dating showed the two clays to be very recent (less than 6300 BP at $z = 7.0$ m for the Tan An clay and 9500 BP at $z = 6.5$ m for the Can Tho clay), the mechanical and physical properties are quite different. Both clays have developed high degree of microstructures, which has been confirmed from CRS tests of very good quality samples. The effects of sampling disturbance on the engineering mechanical properties of the samples retrieved by the two samplers are also studied.

1 INTRODUCTION

Mekong Delta is one of the largest deltas in Asia, which has a huge potential for various industries, especially agriculture. This delta has been formed since Tertiary, but majority of the current delta is covered by Holocene deposit transported by Mekong River after the last glacier (Huu Chiem, 1993). Average water discharge of the river is 1.5×10^4 m/s and average annual suspended load is 1.6×10^8 t (Hori and Saito, 2003). This Holocene deposit created in a very short geological time shows very unique feature. Figure 1 is the distribution of soil status at shallow depth in Mekong Delta (At lat, 2002), which clearly shows the geological history and current natural conditions. Acid-sulfate soil is evidence that the delta was under the sea in transgression period (5000–6000 BP). Fe^{3+} and SO_4^{2-} in the sea water formed pyrite (FeS_2) in the deposit with reduction condition under the mangrove forest. But this marine deposit became land by regression and oxidization of pyrite produced acid sulfate, resulting in acid-sulfate soils widely prevailing in the delta.

As the average elevation of the Mekong Delta is less than one meter, the delta has suffered flooding and salt intrusion for long periods of time. During rainy season, huge area more than a million hectares along the major rivers in the delta is flooded. The soils are transported by the river and sedimented along the river forming fluvial deposits. On the other hand, large tidal range of South China Sea causes salt intrusion, which creates saline soils in the coastal area as shown in Figure 1. According to Hori and Saito (2003), the current Mekong Delta is characterized as mixed energy type delta, of which progression is controlled by wave and tide. But in the beginning of the delta formation, it was supposed to be tide-dominated type delta.

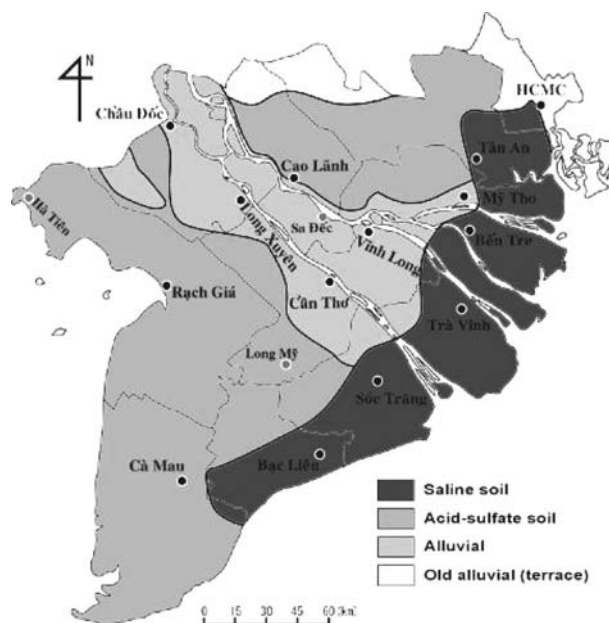


Figure 1. Distribution of soil status in Mekong Delta (At lat, 2002).

From the above mentioned dynamics of Mekong Delta, it can be expected that the formation of the alluvial deposit in the delta has been affected by various factors, flooding, tidal and wave energy, as well as the discharge of the soils and eustacy. This might create quite complicated subsoil conditions.

Man (2003) reviewed more than thirty soil investigation reports on soft clayey soils in Mekong Delta. Figure 2 is the soil profiles at various location of Mekong Delta given in his report, showing very complex and quite deep soft alluvial soils not only in the coastal area but also almost all area in the delta. From the soil investigation reports, Man (2003) compiled the consolidation properties of the soft clayey soils in the delta and showed curious trends in their properties. Figure 3 is one example where overconsolidation ratios (OCRs) decrease with depth and most OCRs are less than unity. From the change of void ratio by recompression to the in-situ effective vertical stress (Figure 4), he attributed the low sample quality to the strange mechanical properties of the soils given in the reports. Disturbance of the sample might be caused by sampling and other processes after sampling. Sampling methods commonly used in Vietnam for relatively deep soft soils is open sampling with Shelby tube, although many literatures have shown the advantage of fixed piston type sampler in the sampling quality compared with the open type sampler especially for soft soils (e.g., Marcuson & Franklin, 1979, Tanaka and Tanaka, 1999). Because of such disturbance commonly observed in the soft clay samples as well as the complicated formation process of alluvial deposit in the delta, mechanical properties of Mekong Delta clay have not been well studied yet.

In order to provide more reliable geotechnical information in Mekong Delta soft alluvial soils, collaboration researches on characterization of soft Mekong Delta clay have been done at two sites, Can Tho and Tan An between Japanese partners (Tokyo Institute of Technology (TIT) and Port and Airport Research Institute (PARI)) and Vietnamese partners (Ho Chi Minh City University of Technology (HCUT), Transport Engineering Design Incorporate (TEDI) and Union Survey Cooperation (USCo), Asian Institute of Technology (AIT)). In this collaborative research, clay samplings with Japanese type thin-walled tube sampler with fixed piston (JFP) and Shelby tube samplers were conducted at the two sites. Consolidation and strength properties of the clay sampled by the two methods were obtained from the laboratories in PARI, TEDI and HCUT using the common procedures and equipment in Japan and Vietnam and the test results obtained in Japan and Vietnam were compared. Various in-situ tests were also conducted at the site, such as piezocone penetration test, Marchetti's flat dilatometer test, field vane test. This report describes the results of the field tests and the laboratory tests done at PARI.

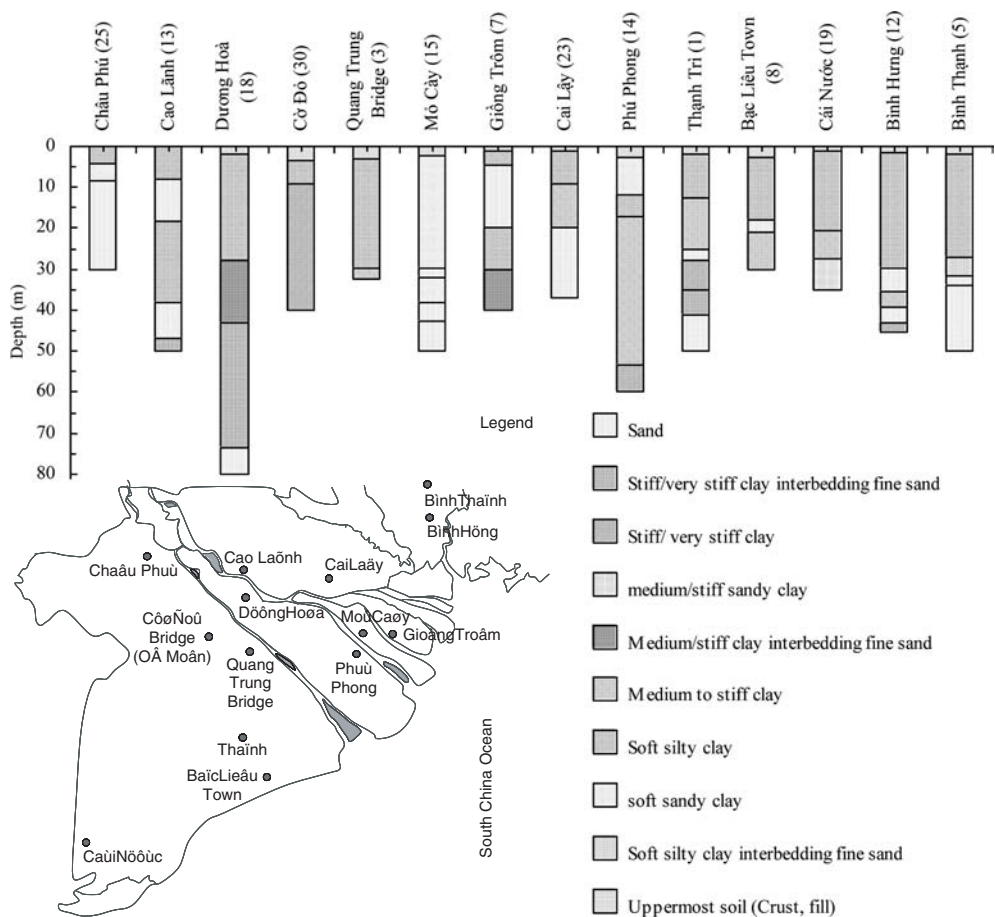


Figure 2. Soil stratigraphy of some boreholes in Mekong delta (after Man, 2003).

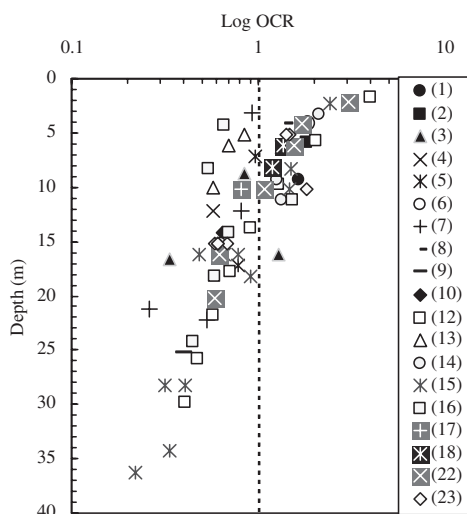


Figure 3. Overconsolidation ratios reported on clay samples at various sites in Mekong Delta (after Man, 2003).

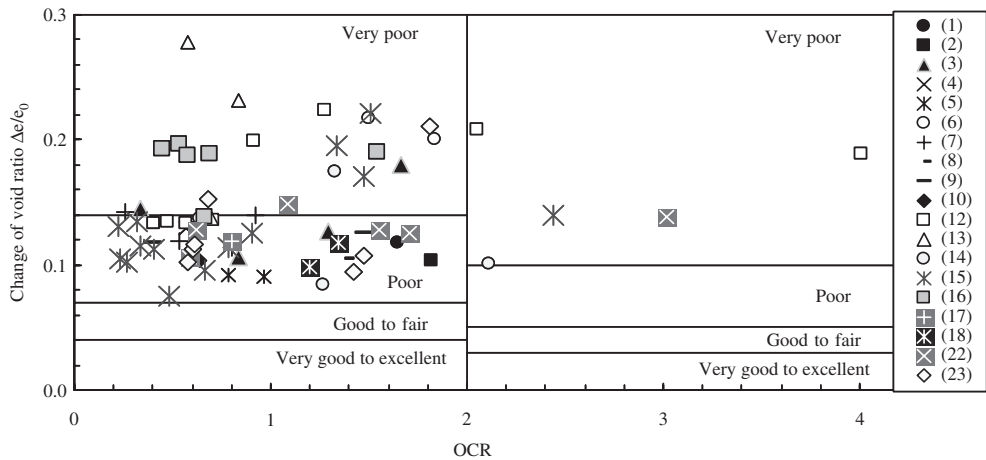


Figure 4. Changes of void ratio due to recompression to the effective overburden stress obtained in oedometer tests on clay samples at various sites in Mekong Delta (after Man, 2003).

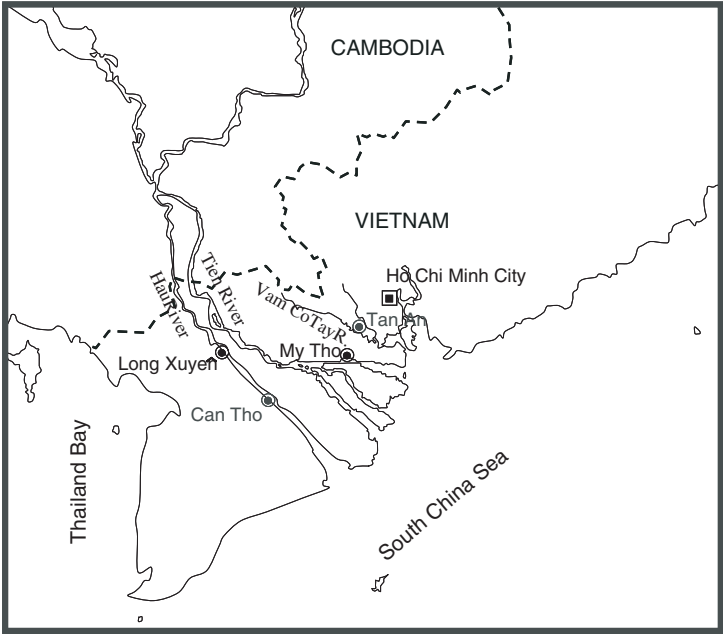


Figure 5. Location of Tan An and Can Tho.

2 TEST SITES AND SOIL PROFILES

2.1 Tan An site

Tan An is located about 50 km southwest from Ho Chi Minh City (Figure 5). As seen in Figure 1, the saline soils indicates the effect of salt intrusion in this area, although the Tan An is more than 30 km from the coastal line. The test site is a swamp area beside Bam Co Tay River. As Bam Co Tay River is not part of Mekong River. Meandering shape suggested the relatively low discharge and sedimentation of fine particles on the riverside.

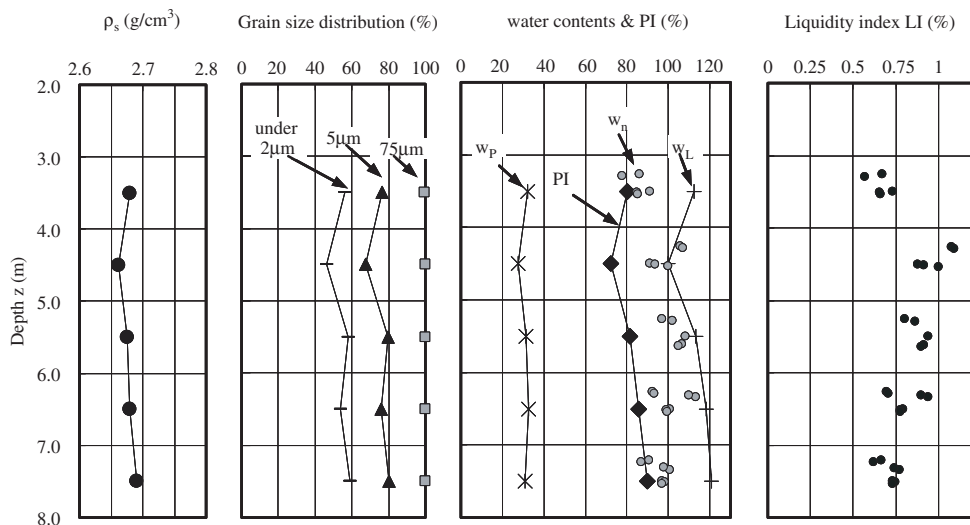


Figure 6. Profile of physical properties at Tan An site.

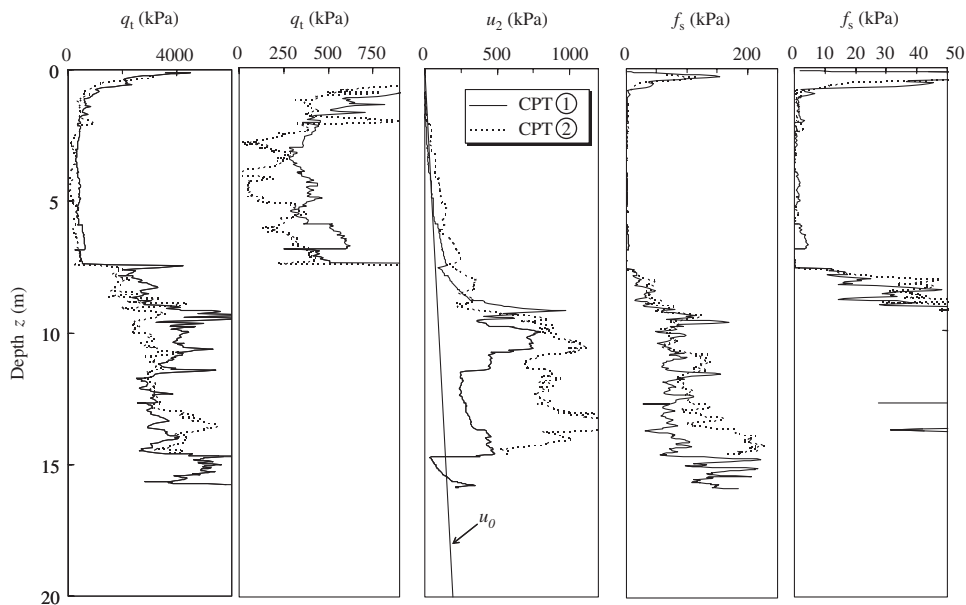


Figure 7. Results of CPTU at Can Tho site.

Figures 6 and 7 show the profile of physical properties and results of piezocone test (CPTU) results at the Tan An site respectively. At the Tan An site, 1m thick fill material was placed on very soft clay of about 6.5 m thickness. As the stiffness of the layer underlying the soft clay was quite high (see Figure 7), the sampling tubes could not be inserted using the sampling equipment employed at the Tan An site.

2.2 Can Tho site

As seen in Figure 5, Can Tho City is about 160 km southwest from Ho Chi Minh City situated at the right bank of Hau River, one of the major streams of Mekong River in Vietnam. The field tests were carried out at a working yard for the construction of Can Tho Bridge, a cable stay bridge crossing Hau

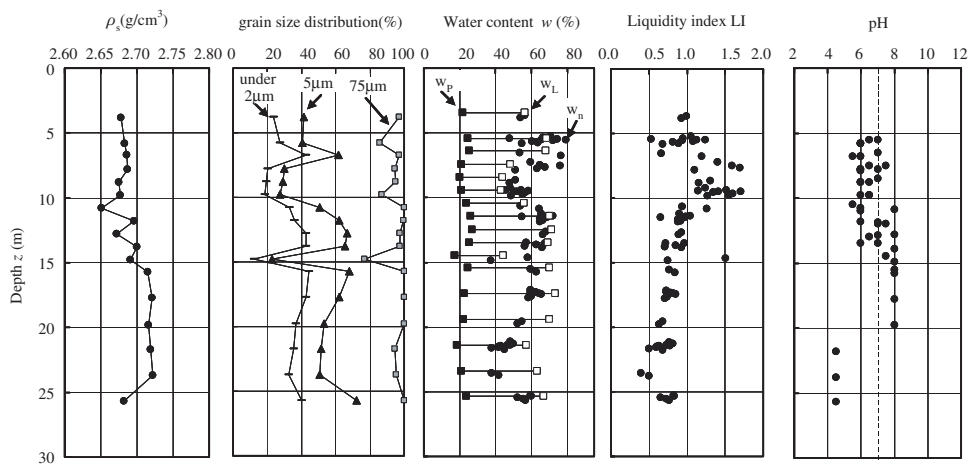


Figure 8. Profile of physical properties and pH at Can Tho site.

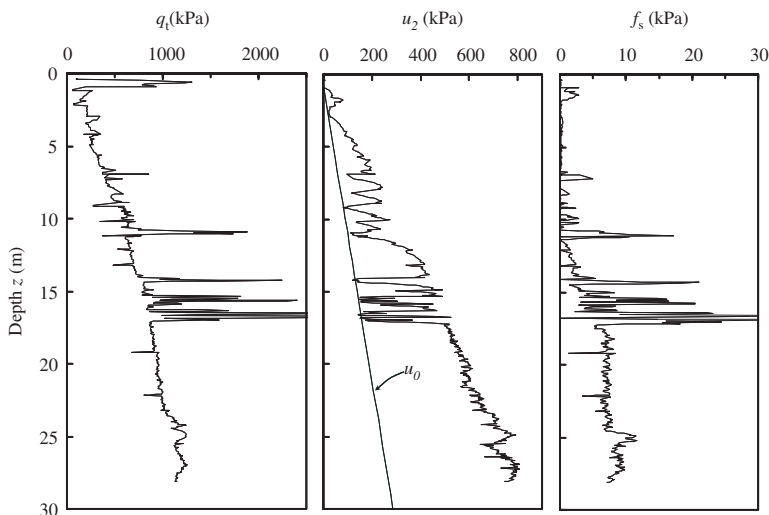


Figure 9. Results of CPTU at Can Tho site.

River. The working yard for the tests was the left bank of the river, which is on the opposite side from Can Tho City. The site was originally flooding area beside the levee. The elevation of the site was raised from 1.4 m to 2.3 m by filling, but it subsided to about 2 m due to consolidation. Although the location is about 100 km upstream from the river mouth, the river water level oscillates daily due to the large tidal range of South China Sea. But large discharge of the Mekong Rivers prevents the saline intrusion along the major river sides and the ample supply of fresh water including flood has created the better ground condition for agriculture in the river side area than the surrounding area with acidic soils (see Figure 1).

Profiles of physical properties and CPTU results are shown in Figures 8 and 9, respectively. At the Can Tho site, sampling was done to the depth of 26 m which was the maximum depth that JFP sampler can be inserted by the water pump used. From the trend of cone resistance (q_t), and pore water pressure measured behind the cone tip (u_2), both of which increases with depth, the deposit is considered typical normally consolidated soft clay. But sudden increases of q_t and drops of u_2 from the trends indicate very shallow sand layer or intermediate soils with high permeability.

From carbon isotope C^{14} dating shown in Table 1, it can be said that all soils investigated at the Can Tho site can be categorized as Holocene deposits. Although the dated year may not give the correct deposition

Table 1. Results of carbon isotope C^{14} dating.

Tan An site			Can Tho site		
Depth (m)	Year (BP)	dz/dt (mm/y)	Depth (m)	Year (BP)	dz/dt (mm/y)
4.0	1545 ± 40	1.9	5.8	3739 ± 30	1.3
7.0	6346 ± 35	0.6	13.8	4100 ± 30	22.2
			19.7	6385 ± 30	2.6
			23.7	8146 ± 35	2.3
			25.6	9498 ± 35	1.4

Table 2. In-situ tests conducted.

Type of test	Can Tho site	Tan An site
Piezcone penetration test (CPTU)	Yes	Yes
Marchetti's dilatometer test (DMT)	Yes	Yes
Field vane test (FV)	Yes	No
Hydraulic fracturing test (HFT)	No	Yes
Multi-point spectra analysis of surface wave (SASW)	No	Yes

Table 3. Laboratory tests conducted at PARI.

Type of set	Can Tho clay		Tan An clay	
	JFP	Shelby	JFP	Shelby
Index test	Yes	No	Yes	Yes
pH measurement	Yes	Yes	No	No
Incremental loading oedometer test	Yes	Yes	Yes	Yes
CRS consolidation test	Yes	Yes	Yes	Yes
Unconfined test with suction measurement	Yes	Yes	Yes	Yes
Constant volume direct shear test (CVDST)	Yes	No	No	No
CK_0 UC and CK_0 UE triaxial tests	Yes	No	No	No
Laboratory vane test	Yes	Yes	No	No

period, the deposition speed in terms of the current thickness is at least more than 2.6 mm/year, which is much higher than that of Tan An clay (1.0 mm/y). From this dating and the CPTU results (Figure 9) it can be expected that a rapid deposition took place at the depth from 5 m to 17 m.

3 TEST PROGRAMS

3.1 In-situ tests and samplings

In-situ tests conducted at the two sites are listed in Table 2. At both sites, soft soils were retrieved by two sampling methods, i.e., Japanese type thin-walled tube sampler with fixed piston (JFP) and Shelby tube samplers. At Tan An site, both types of sampling were conducted from 3.5 m to 7.5 m every 1 m. At Can Tho site, JFP was conducted to 26 m depth, every 1 m from 3 m to 16 m and every 2 m from 16 to 26 m. Shelby tube sampling was done for the shallower depth. Originally Shelby tube sampling was planned from 5 m to 9 m with 1 m interval, however the open type sampler could not retrieve the soil at some depths. Therefore the Shelby tube sampling was continued to 13 m depth to obtain five samples.

3.2 Laboratory tests

Laboratory tests done at PARI for the clay samples collected by Japanese type fixed piston thin wall sampler (JFP) and Shelby tube sampler at the two sites are listed in Table 3. As described in the subsequent section, there were many holes in the samples of Tan An clay, which made it difficult to measure

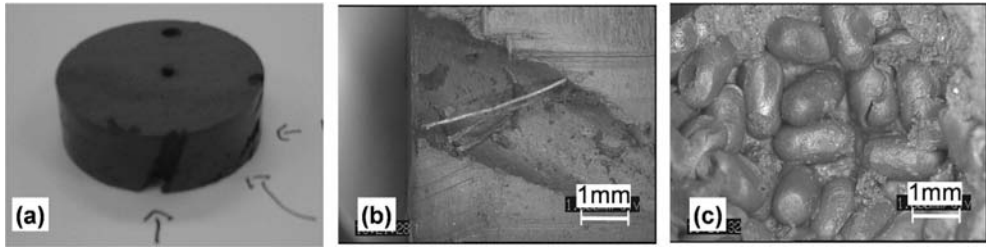


Figure 10. evidences of bioturbations in Tan An clay: (a) holes probably made by micro-organism; (b) plants; (c) evidence of organism.

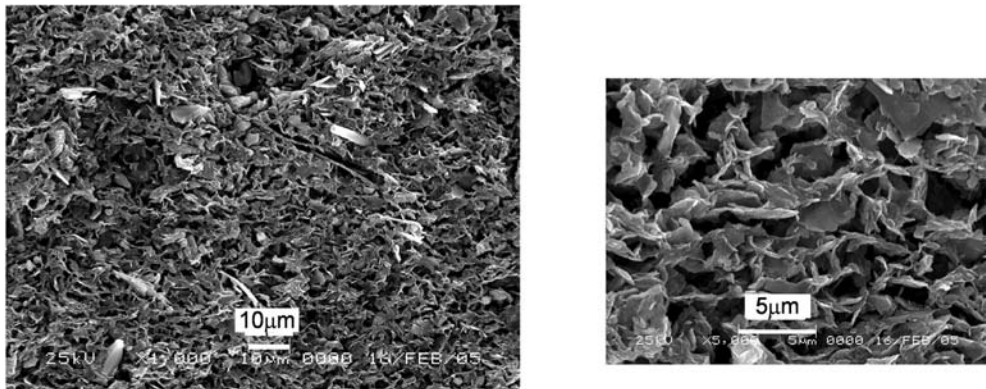


Figure 11. Scanning electron micrographs of Tan An clay $z = 6.5$ m.

excess pore-water pressure, K_0 consolidated undrained shearing tests and constant volume direct shear tests were not conducted for Tan An clay. Incremental loading oedometer tests of reconstituted samples were also carried out to obtain the intrinsic compression behavior of the two clays.

4 MATERIAL COMPOSITIONS AND STRUCTURES

4.1 Grain size distributions

As seen in Figures 6 and 8, grain size distributions of Tan An and Can Tho sites are very different. Tan An clay has large fraction of fine grain (under $2\ \mu\text{m}$) of more than 50% with relatively uniform distribution in the sampling depths from 3.5 m to 7.5 m. While for Can Tho clay, the grain sizes are larger than those of Tan An clay. The grain size distribution fluctuates at the shallower depth down to 16 m, which corresponds to the variation of q_t and u_2 measured by CPTU (see Figure 9). It should be noted that some local variation of soil profiles estimated by CPTU, e.g., $z = 15\text{--}17$ m, cannot be captured by the grain size analyses and index tests done with the samples collected at 1 m depth interval above 16 m and 2 m depth interval below 16 m.

4.2 Structures

4.2.1 Macrofabric and microfabric of Tan An clay

Due to very recent and shallow deposition, the Tan An clay shows some evidences of bioturbation as shown in Figures 10. The most specific feature of the Tan An clay is randomly distributed holes, which have been probably burrowed by micro organisms. Although the Tan An clay has quite uniform microfabric with very thin particles with large pores (see Figure 11) and uniform composition in the depth investigated (see Figure 6), results of CPTU are very sporadic and less repeatable. This can be attributed to the holes in the clay.

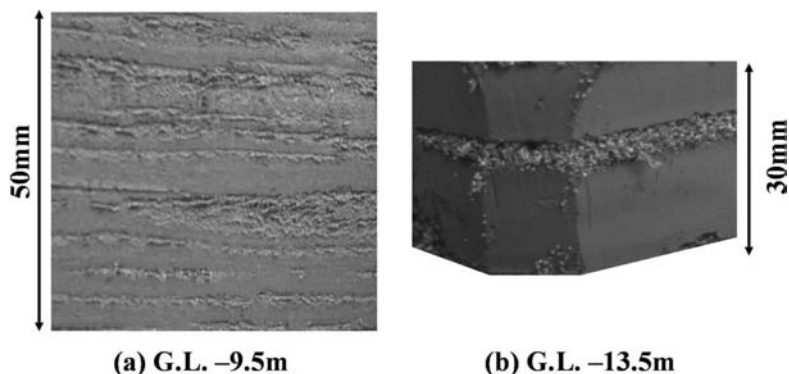


Figure 12. Very thin sand seams in Can Tho clay.

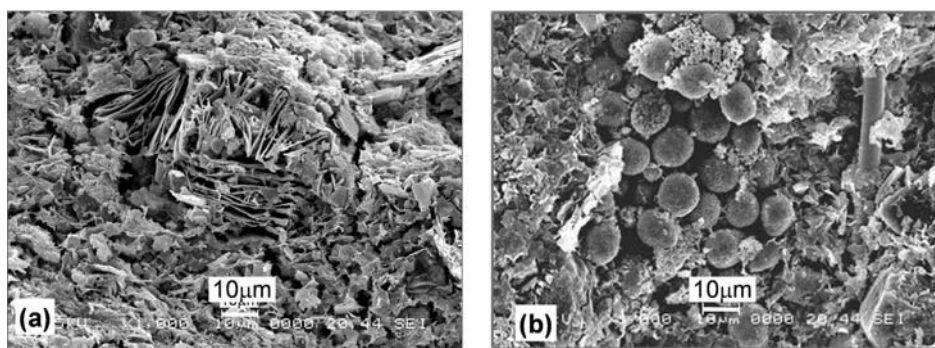


Figure 13. Scanning electron micrographs of Can Tho clay: (a) $z = 12.5$ m; (b) $z = 21.5$ m.

4.2.2 Macrofabric and microfabric of Can Tho clay

As seen in Figures 8 and 9, the Can Tho clay shows strong heterogeneity with depth. Clay dominates the composition but very thin sand seams are occasionally included in the clay as shown in Figures 12. The thickness and composition of sand seams or intermediate soils depends on the level of flooding and discharge rate of Hau river. Figure 13(a) shows relatively larger clay particles and smaller pore size of Can Tho clay than Tan An clay. From SEM image of the clay at depth of 21.5 m (Figure 13(b)), pyrite can be observed, which suggests that the clay at this depth sedimented in marine environment. This can also attributes to the relatively high acidity of the clay at deeper depth shown in Figure 8.

5 STATE AND INDEX PROPERTIES

5.1 Atterberg limits and activity

At the Tan An site, Atterberg limits and plasticity indices are quite uniform in the very soft clay layer as seen in Figure 6. From the plasticity chart on various clay in the world (Figure 14), the Tan An clay can be categorized as high plastic clay with $w_L = 100\text{--}120\%$ and $PI = 72\text{--}90$, well above A-line in the chart. As for the Can Tho clay, the data points are also well above A-line, but w_L and PI range $44\text{--}70\%$ and $22\text{--}50$ showing relatively low plasticity.

Figure 15 shows the activity of the two clays. Activity of the Tan An clay is very high with average $A = 1.5$ and the Can Tho clay has intermediate activity about $A = 1.0$. The Can Tho clay has larger variation of activity than the Tan An clay.

5.2 Natural water contents and liquidity indices

Natural water contents (w_n) of Tan An clay are very high ranging from $80\text{--}110\%$, but the liquidity indices (LI) are less than unity with a few exception as shown in Figure 6. Can Tho clay has relatively low natural

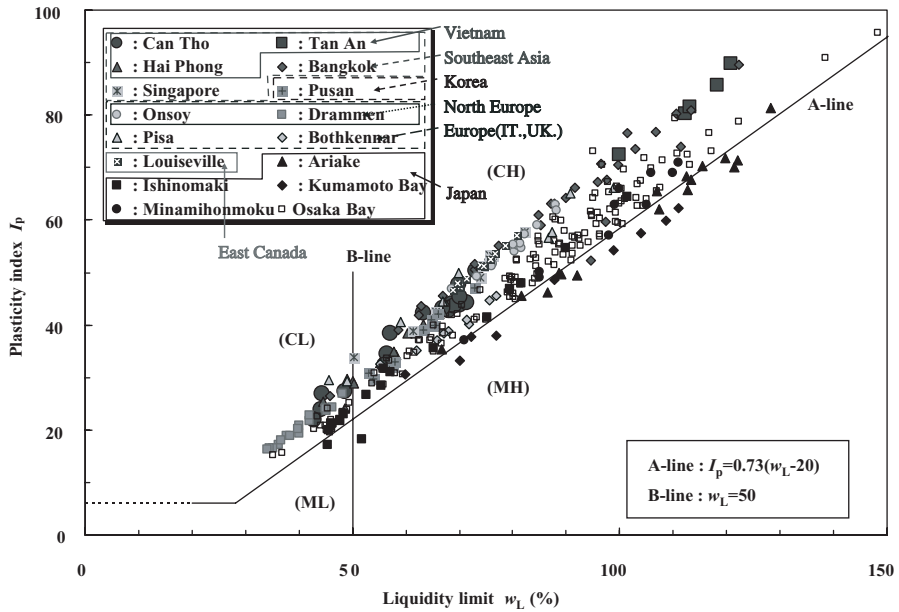


Figure 14. Plasticity chart.

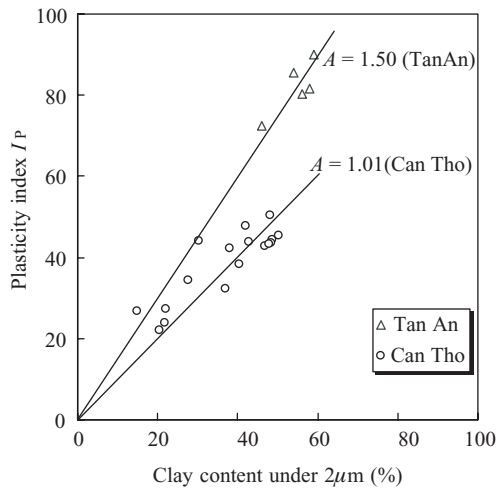


Figure 15. Activity of three clays in Vietnam.

water contents with large variation from 38% to 79% (Figure 8). Liquidity indices vary widely especially at depths shallower than 15 m. At the depth from 7 m to 10 m where very low plastic soils are deposited, LIs are well above unity, 1.8 as maximum and LIs below 16 m are all less than unity. It should be noted that the data on w_n are obtained from two bore holes, one for JFP sampling and one for Shelby tube sampling, which are only 10 m apart, showing very large fluctuation of the state of soil in the vertical direction. Figure 16 shows initial water contents of the specimens used for various laboratory tests. The fluctuation of water contents with depth is more prominent, even at very small depth difference. As the liquidity index was calculated using Atterberg limits obtained at the closest depth, this might cause further variation of LI in the vertical variation.

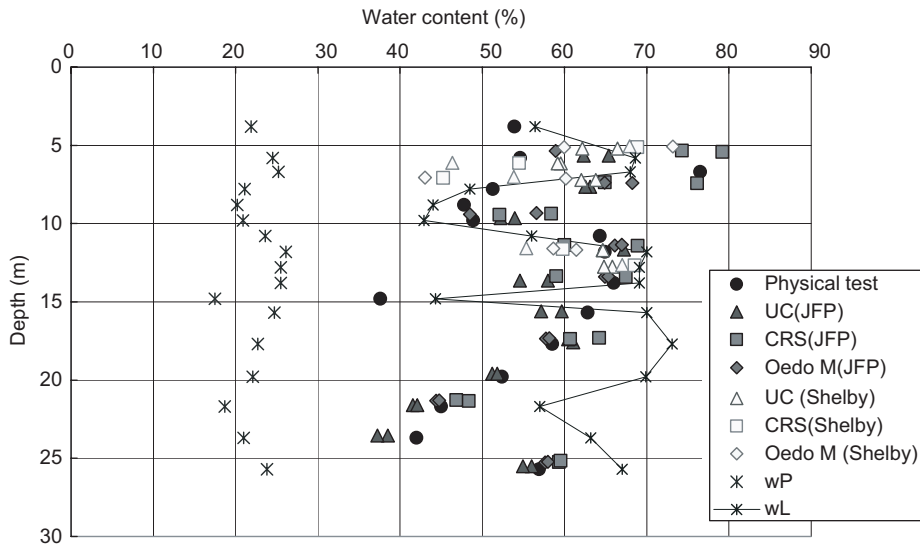


Figure 16. Initial water contents of samples for various tests: Can Tho clay.

6 ENGINEERING PROPERTIES

As reported by Man (2003), previous soil investigations on Mekong Delta clays produced strange results on consolidation properties from the oedometer tests, such as decreasing OCR with depth to very low value far less than unity and very large compression due to reloading to the effective vertical stress (Δe) as shown in Figures 3 and 4. This could be attributed to disturbance caused by various processes of soil sampling and testing. In order to investigate the sensitiveness of Mekong Delta clay to the sampling disturbance, various mechanical tests were performed on samples retrieved by JFP and Shelby.

6.1 Consolidation tests

6.1.1 Compressibility

Figure 17 shows $e-\log \sigma'_v$ relations measured by constant rate of strain consolidation test (CRS) for Can Tho clay retrieved at various depth by the JFP sampler. Except for the sample at the depth of 9.4 m, S-shape curves with clear bent were observed, which is typical compressibility of natural clay but has rarely been observed in the soil investigation of Vietnamese clay (Man, 2003).

The $e-\log \sigma'_v$ relations measured by CRS test for the samples retrieved by JFP sampler and Shelby tube sampler (hereafter called JFP sample and Shelby sample respectively) are compared both for the Tan An clay and the Can Tho clay in Figures 18. For the Tan An clay with high plasticity, there is less difference in the $e-\log \sigma'_v$ curves for the two samples at the similar depth. Both samples at the shallower depth show gradual decrease of void ratio with $\log \sigma'_v$ and those at the deeper depth show sharp decrease of void ratio after the consolidation yield stress (Figure 18(a)). While for the Can Tho clay with low plasticity, the shapes of $e-\log \sigma'_v$ curves are different in JFP and Shelby samples, sharp decrease after the yield stress in the former and gradual decrease in the latter (Figure 18(b)). From Figures 18 it can be confirmed that sampling disturbance depends not only on sampling methods but also on type of clay.

Figure 19 shows the variation of compression index C_i with consolidation pressure, that is the slope of $e-\log \sigma'_v$ curves observed in CRS tests for the JFP samples of Can Tho clay. The C_i shows sharp increase to the maximum value just after the yield stress and then gradually decreases to almost a constant value at the stress well above the yield stress. Following the method of Tan et al. (2003), the index C_{C1} and C_{C2} were obtained, where C_{C1} is the maximum C_i and C_{C2} is the C_i at $\sigma'_v = 1,000$ kPa. These indices and their ratio C_{C1}/C_{C2} are plotted against depth in Figure 20. At the Tan An site (Figures 20(a) and (b)), the observed C_{C2} are almost constant with depth, $C_{C2} = 0.66-0.96$, regardless of sample types, while the compressibility just after yield stress, C_{C1} , dramatically changes at the depth of 6 m; $C_{C1} = 0.75-1.47$ and $C_{C1}/C_{C2} = 1.1-1.7$ for the shallower depths and $C_{C1} = 2.28-4.88$ and $C_{C1}/C_{C2} = 2.3-6.2$ for the deeper depths. From the

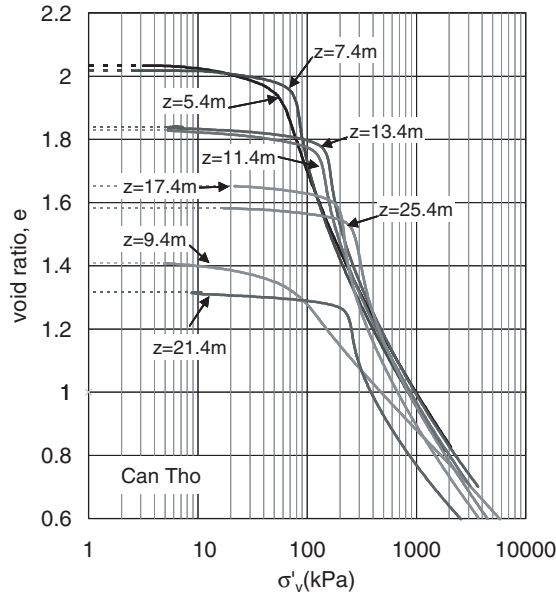


Figure 17. $e-\log \sigma'_v$ curves of the samples retrieved by JFP: Can Tho, CRS.

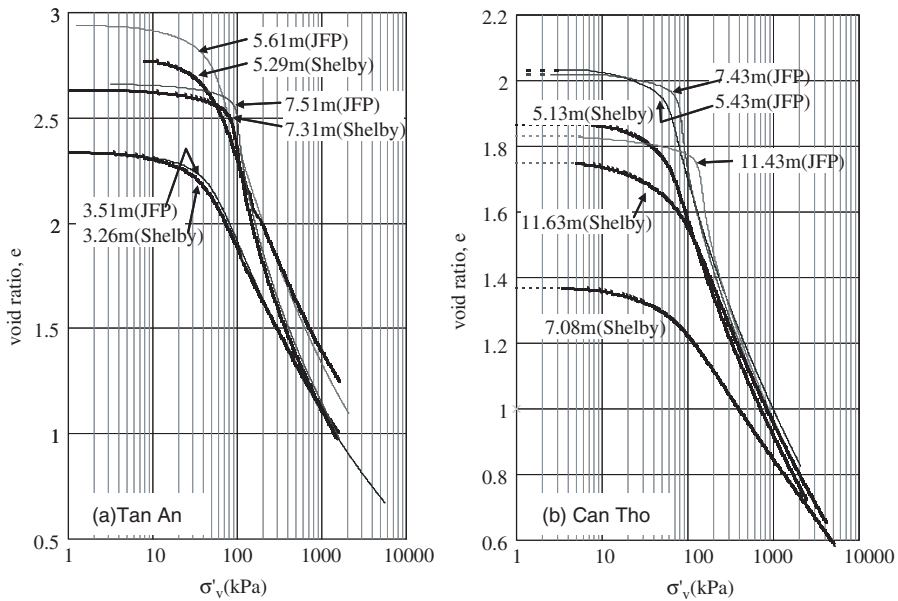


Figure 18. Comparison of $e-\log \sigma'_v$ curves of samples retrieved by JFP and Shelby sampler.

difference of microstructuration at the shallower and deeper depths and the results of C_{14} dating shown in Table 1, it can be inferred that deposition period might not be proportional to depth and there should be a gap of the period at a depth of about 6 m. From the C_{C1}/C_{C2} ratio, the difference between JFP and Shelby samplers can be seen for the deeper samples but this difference is not clear for the shallow very young soil.

At the Can Tho site, C_{C2} and C_{C1} values obtained from JFP samples are 0.39–0.53 and 0.46–3.3 as shown in Figure 20(c). C_{C1}/C_{C2} ratio of the Can Tho clay ranges from 1.2 to 6.7 showing the tendency of increasing with depth (Figure 20(d)). This trend becomes more clear if the data at the depth with low plasticity index are excluded, e.g., the depth about 9.4 m. Although the results on Shelby tube samples

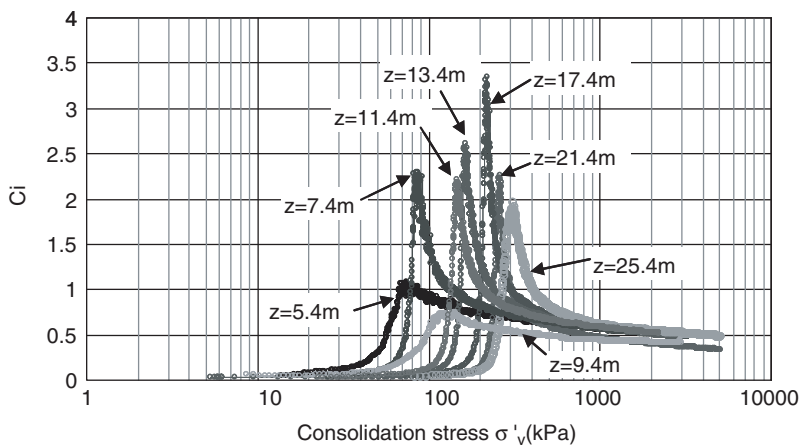


Figure 19. Variation of compression index C_i with σ'_v observed by CRS: JFP samples, Can Tho clay.

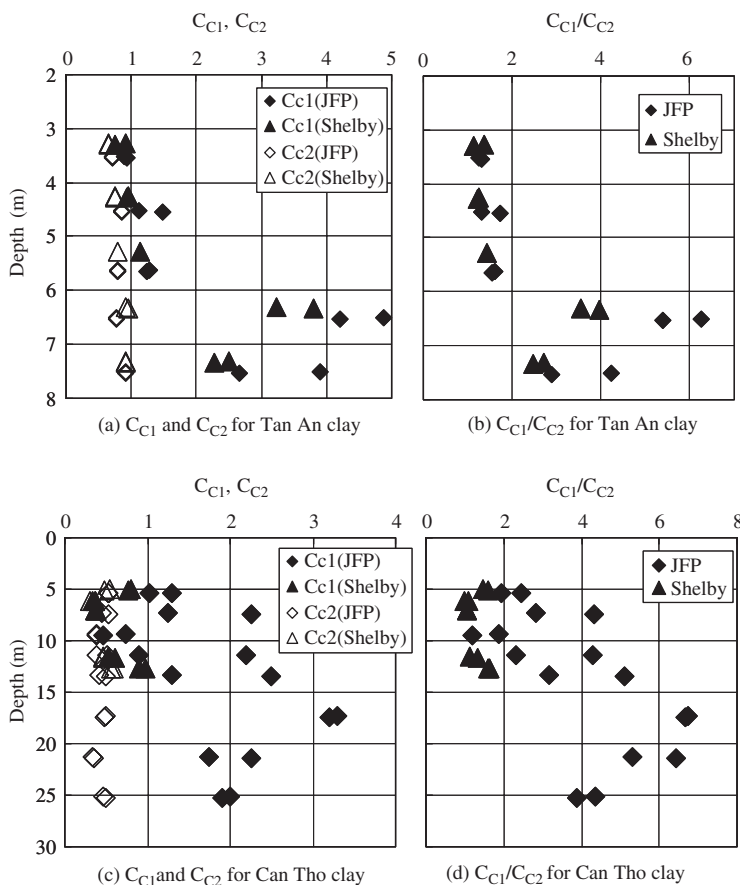


Figure 20. Variation of C_{C1} , C_{C2} , C_{C1}/C_{C2} with depth for JFP and Shelby samples: CRS.

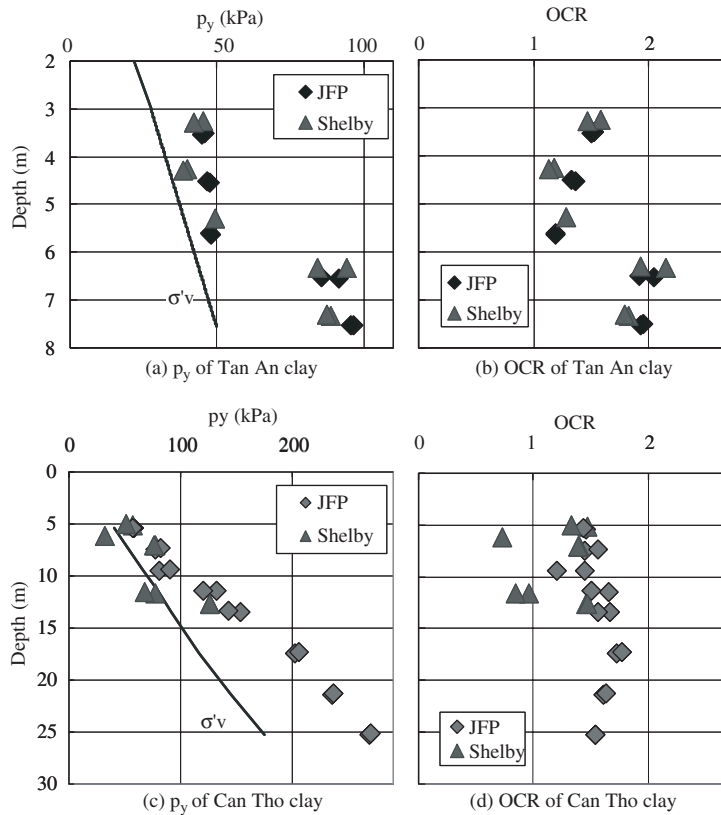


Figure 21. Variation of p_y and OCR obtained from CRS tests for Tan An and Can Tho clays.

are limited in the shallower depth, the distinct difference between JFP and Shelby tube samples can be confirmed from the ratio of C_{C1}/C_{C2} .

The largest C_{C1}/C_{C2} ratios observed for the Tan An and Can Tho clays, more than 6, are even larger than the ratio of Pleistocene marine clay at the New Kansai Airport site, which is well known clay with high degree of structures (Tanaka et al., 2003), suggesting the very high structuration of the two clays, despite very recent deposition about 6,000 BP as shown in Table 1.

6.1.2 Consolidation yield stresses and OCRs

Consolidation yield stress (p_y) and overconsolidation ratio (OCR) obtained from the CRS tests are shown in Figures 21. The OCR of Tan An clay (Figure 21(b)) changes at the depth of 6 m, lower OCR (1.1–1.6) at the shallower depth and higher OCR (1.8–2.2) at the deeper depth. The difference of OCR between the two samples is not noticeable for the Tan An clay.

For the JFP samples, the p_y of Can Tho clay increases almost linearly with depth, resulting in relatively constant OCR with depth ranging from 1.5 to 1.8, except at $z = 9.4$ m (Figures 21 (c) and (d)). While for the Shelby tube sample, there is no clear trend of p_y and the OCRs are similar to those of the JFP samples at some depths, but much smaller than JFP at some other depths, even less than 1. From the above comparisons, it can be said that very small OCRs reported in the previous soil investigations (see Figure 3) could be partly attributed to the poor sampling quality.

6.1.3 Intrinsic compression behavior

For some depths, incremental loading oedometer tests were conducted for the reconstituted sample. The specimen was prepared from the slurry with water content of 1.5 times the liquid limit. For the Tan An clay, the slurry was first consolidated in the preconsolidation mold with $\sigma'_{v0} = 50$ kPa and then the preconsolidated clay was inserted into the oedometer ring. While for the Can Tho clay, as the preconsolidated clay block is too soft to stand without confinement, the slurry was preconsolidated in

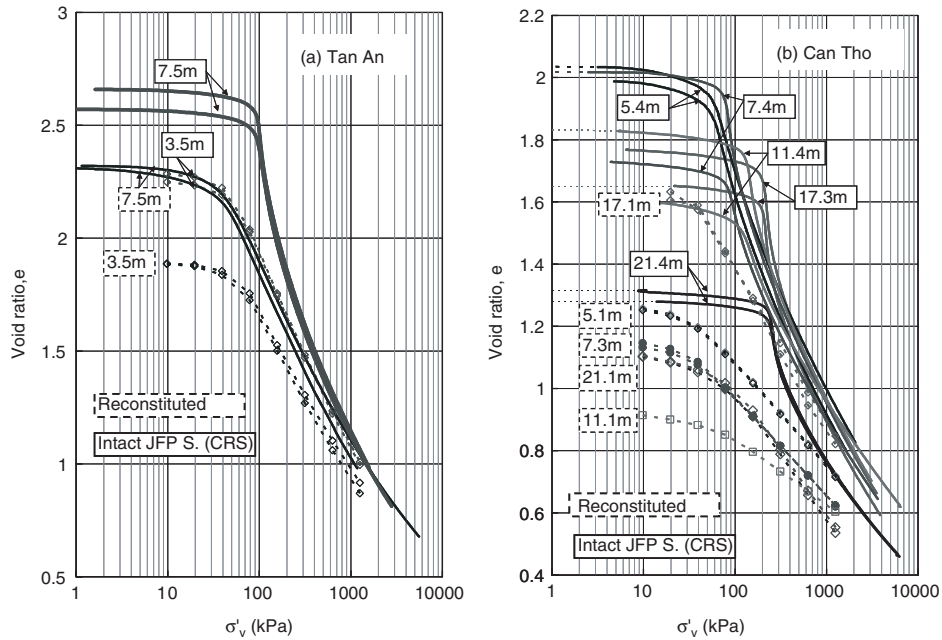


Figure 22. Intact and intrinsic compression curves of Tan An and Can Tho clays: JFP samples.

the oedometer mold with $\sigma'_{v0} = 20$ kPa. The e - $\log \sigma'_v$ curves of reconstituted samples are compared to those of JFP samples in Figures 22. For the Tan An clay, the e - $\log \sigma'_v$ curves of reconstituted and JFP samples tend to merge at higher stress. While for the Can Tho clay, in general there are large differences between reconstituted and JFP samples, no clear tendency of convergence of the curves at the higher stress. The difference of void ratio at the higher stress for the reconstituted samples is larger than the JFP samples, especially for the samples at depths shallower than 11.5 m. It can be also confirmed from Figure 22 (b) that the intrinsic properties of Can Tho clay changes with depth significantly.

In-situ void index (I_{v0}) was evaluated using the normally consolidated line in the e - $\log \sigma'_v$ curves of reconstituted samples as the intrinsic compression curves, following the definition proposed by Burland (1990). The I_{v0} is plotted to in-situ effective vertical stress with intrinsic compression line (ILI) and sedimentary compression line (SCL) proposed by Burland (1990) in Figure 23. The I_{v0} of Tan An clay at $z = 3.5$ m is in the middle position between ICL and SCL and that at $z = 7.5$ m is on or slightly below SCL. This difference of I_{v0} gives large difference of structuration as shown in Figures 18, 20 and 21. The I_{v0} indices of Can Tho clay at the shallower depths are well above SCL and those at the deeper depths are on or slightly over SCL as shown in Figure 23. However, degrees of structuration in terms of C_{C1}/C_{C2} and OCR are increasing with depth and less depth dependent respectively (see Figures 20(b) and 21(b)). Very large I_{v0} values might be attributed to the heterogeneity in the vertical direction as shown in Figures 12. The reconstituted clay was prepared from a 50 mm height sample. If the coarse and fine grain materials from different thin layers are mixed, the mixture becomes a well graded material of which intrinsic properties is different from the natural one with lower void ratio.

6.1.4 Coefficient of consolidation

Figure 24 shows the variation of coefficient of consolidation (c_v) with consolidation stress σ'_v obtained from CRS tests on JFP samples of Can Tho clay. Reliable c_v could not be obtained from Tan An clay, which is probably because of the randomly distributed holes in the sample (Figure 10). The c_v value drastically drops at the stress near p_y and then increase after the minimum value. Figure 25(a) shows the depth variations of averaged c_v values from the in-situ vertical stress σ'_{v0} to the p_y , which is denoted by c_{v1} , and c_v values at $\sigma'_v = 1000$ kPa, denoted by c_{v1000} . The ratios of c_{v1}/c_{v1000} are shown in Figure 25(b). For the JFP samples, c_{v1} is very much larger than c_{v1000} , except for the samples at $z = 9.4$ m of which c_{v1} is smaller than c_{v1000} . While for the Shelby tube samples, there is no consistency of relative magnitude between c_{v1} and c_{v1000} or the ratio c_{v1}/c_{v1000} indicating the large variation of sample quality.

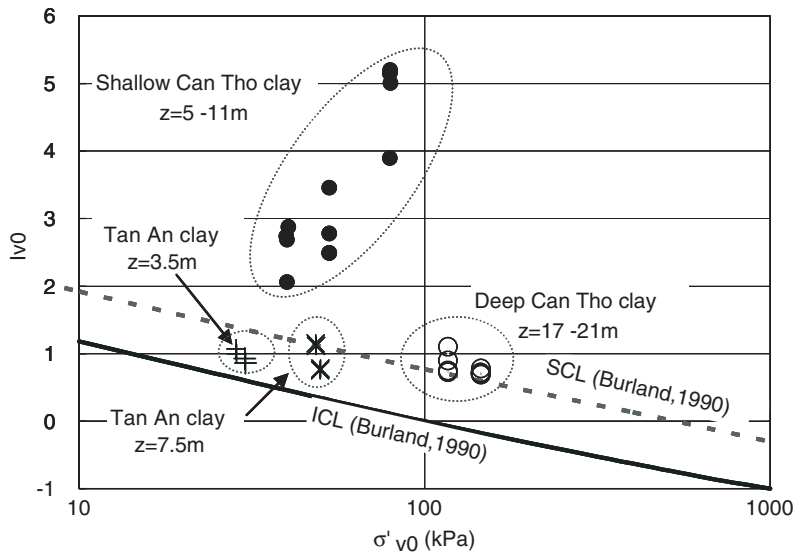


Figure 23. Relationship between void index I_{v0} and σ'_{v0} for Tan An and Can Tho clays.

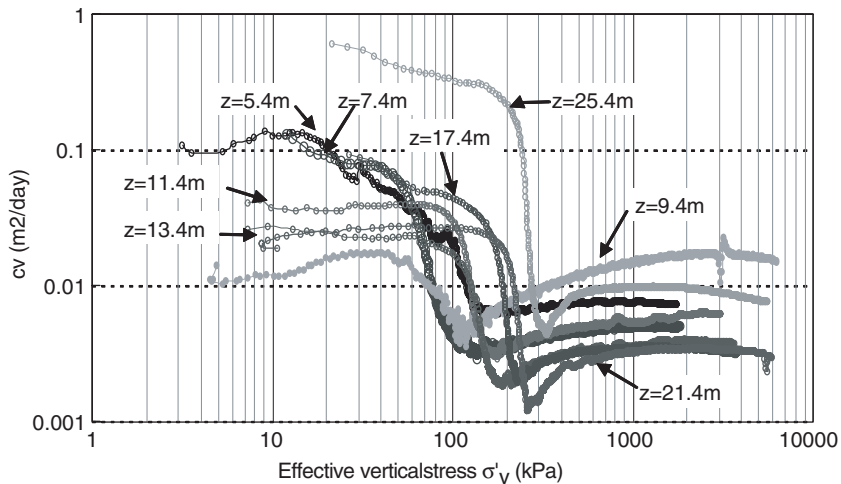


Figure 24. Variation of c_v with σ'_v observed by CRS: JFP samples, Can Tho clay.

6.1.5 Change of void ratio by recompression to the in-situ vertical stress, Δe

Change of void ratio or volumetric strain caused by the recompression to in-situ vertical stress, σ'_{v0} , can be used as indicators of sample quality (Andresen and Kolstad, 1979; Lacasse and Berre, 1988; Lunne et al, 1997). The change of void ratio normalized by the initial void ratio, $\Delta e/e_0$, are plotted to the depth of the sample in Figure 26. For the Tan An site, the $\Delta e/e_0$ values range from 0.033 to 0.053 for the samples shallower than 6 m, except for the Shelby tube sample at $z = 4.3$ m, while for the samples deeper than 6 m, the values range from 0.13 to 0.35. Neglecting the exception at $z = 4.3$ m, it can be reconfirmed that the sample quality difference between the two types of sampler are not significant for the very soft, less structured clay at the shallower depth compared to that for the relatively stiff clay with structuration at the deeper depth as discussed for the C_{C1}/C_{C2} in Figure 20(b).

For the Can Tho site, the $\Delta e/e_0$ values of JFP samples are from 0.017 to 0.037 within the range of very good or excellent quality, defined by Lunne et al. (1997) except for the samples at $z = 9.4$ m with

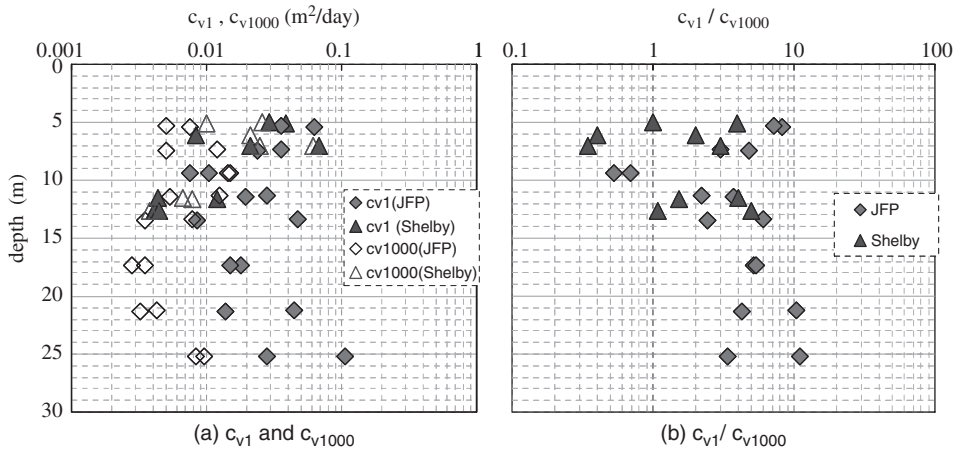


Figure 25. Variation of c_{v1} , c_{v1000} and c_{v1}/c_{v1000} with depth for JFP and Shelby samples: CRS, Can Tho clay.

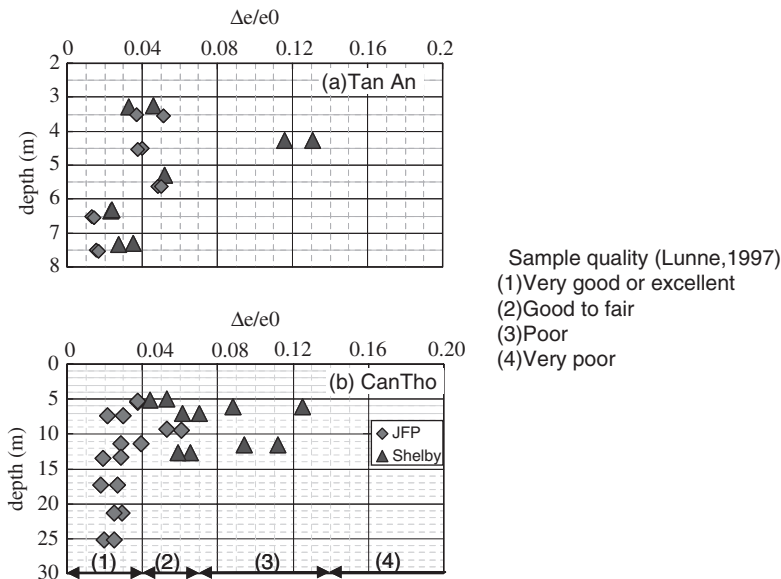


Figure 26. Sample quality in terms of compression due to reloading to σ'_v : CRS.

$\Delta e/e_0$ values of about 0.6 (Figure 26(b)). Comparing Figure 26 with the figures showing the sample quality of natural soils, Figures 20 (b) and (d) on C_{C1}/C_{C2} , Figures 21(b) and (d) on OCR, and Figure 25(b) on c_{v1}/c_{v1000} , it can be confirmed that the $\Delta e/e_0$ value is a good indicator of the sample quality and the samples with the $\Delta e/e_0$ values over 0.07, corresponding to poor quality (Lunne et al, 1997), might yield very low OCR even less than 1 and relatively low C_{C1}/C_{C2} values. It should be noted that the samples could not be retrieved using the Shelby tube sampler at the depth of lowest plasticity index, about 8–10 m.

6.2 Unconfined compression tests

Axial stress–strain curves obtained by unconfined compression (UC) tests are shown in Figure 27. Unconfined compression strength, q_u , residual suction normalized by in-situ vertical stress (σ'_{v0}) and normalized undrained strength, $(q_u/2)/\sigma'_{v0}$ measured by UC tests are summarized in Figure 28. The suction of the test specimen was measured by using the way described by Tanaka et al. (2002). The q_u values at the

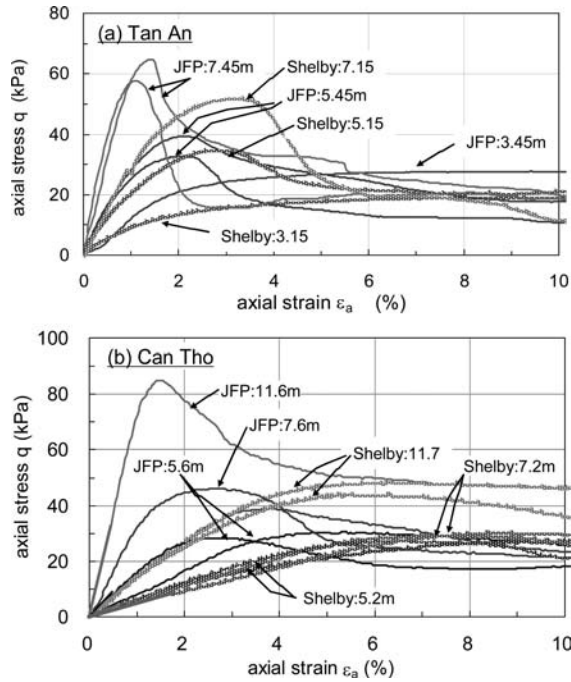


Figure 27. Comparison of axial stress and strain curves of UC tests for JFP and Shelby samplers.

two sites increase almost linearly with depth. At the Tan An site, the normalized strength, $(q_u/2)/\sigma'_{v0}$, tends to increase with depth from about 0.3 at $z = 3.5$ m to 0.6 at $z = 7.5$ m, while at the Can Tho sites no clear trend of the normalized strength with depth but showed a large variation from 0.16 to 0.52. The difference in these parameters between the JFP and Shelby samples are not distinguished clearly. Figures 29 show the relationships between the $(q_u/2)/\sigma'_{v0}$ and the normalized suction. Although the suction measurement for the Tan An clay might not be so reliable because of the holes in the sample, a trend of increasing $(q_u/2)/\sigma'_{v0}$ with the normalized suction can be seen in Figure 29(a). The more apparent trend can be seen between the q_u and the residual suction until the normalized suction becomes about 0.2 for the Can Tho clay. But the normalized strengths are almost constant about 0.4 for the normalized suction over 0.2. This indicates the residual suction or effective stress of the sample as a quality indicator for the undrained shear strength.

Figure 30 show the failure strains, ϵ_f , and the secant moduli divided by the undrained shear strength, $E_{50}/(q_u/2)$. Because E_{50} depends on strength and consolidation pressures, it is normalized by these parameters and can be used as an indicator of sample quality (e.g., Nakase et al, 1985). From Figures 30 it is also confirmed that at the Tan An site there is less difference between these parameters of the two samples at the shallower depth and clear difference at the deeper depth. The difference becomes much more apparent for the Can Tho site.

6.3 K_0 consolidation tests for Can Tho clay

For the JFP samples of the Can Tho clays, K_0 -consolidation tests were conducted using the triaxial test apparatus (Watabe et al., 2003). The specimen with the dimension of 35 mm in diameter and 85 mm in height was first consolidated with isotropic stress of $\sigma'_{v0}/3$. After this preliminary consolidation, K_0 consolidation was performed maintaining zero horizontal strain till the vertical stress reached three times of σ'_{v0} . Variation of the stress ratio σ'_h/σ'_v with the normalized vertical stress σ'_v/σ'_{v0} in the K_0 consolidation test are shown in Figure 31. At all depths of the samples tested, the stress ratios first decrease with increasing vertical stress and converge to constant values, which can be regarded as K_0 values of normally consolidated conditions, K_{0NC} . The values of K_{0NC} ranged from 0.44 to 0.59, similar to those of normal marine clay deposits in Japan (Watabe et al, 2003). According to Watabe et al. (2003), the structuration of the clay can be distinguished from the variation of the stress ratio with the normalized vertical stress. The stress ratio of the clay with high structuration, such as aged Pleistocene

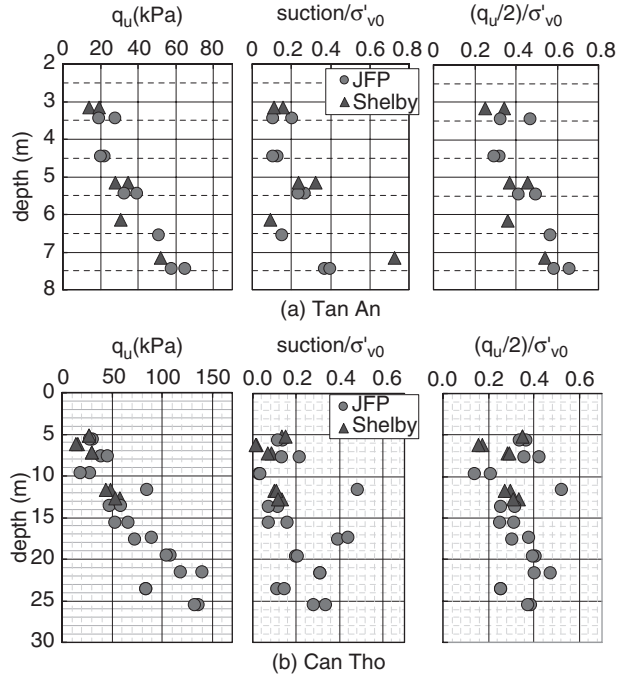


Figure 28. Results of UC tests for JFP and Shelby samplers.

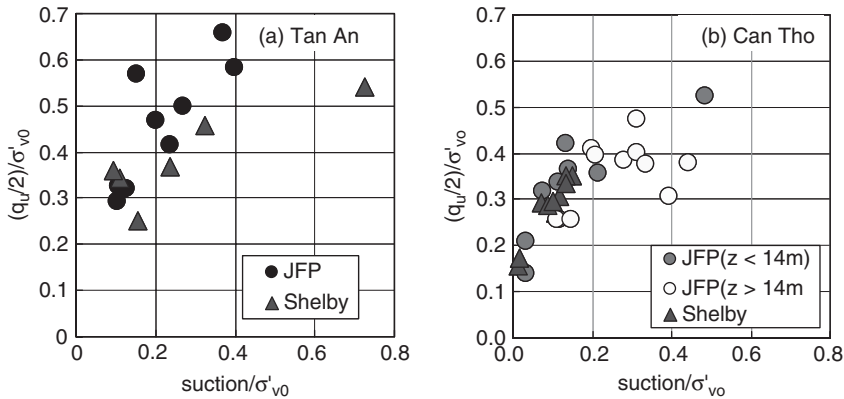


Figure 29. Relationship between suction and undrained strength of UC tests.

clay, decreases the minimum values at the vertical stress slightly above the in-situ stress (σ'_{v0}) and then turn to increases to K_{0NC} . While for the clay with less structuration, such as young normally consolidated clay, sandy soils, and reconstituted clay, the ratio monotonically decreases to K_{0NC} . Comparing Figure 31 with Figure 20(d) the above features can be confirmed for the Can Tho clay. The sample at $z = 9.3$ m with the minimum C_{c1}/C_{c2} ratio shows no minimum values in the variation. On the other hand, the samples at deeper depth with large C_{c1}/C_{c2} ratio show the minimum value and marked increase of the stress ratio.

K_0 consolidated undrained compression and extension tests (CK_0UC and CK_0UE) were also conducted for the specimen anisotropically consolidated with the stress ratio $\sigma'_h/\sigma'_v = K_{0NC}$ and $\sigma'_v = \sigma'_{v0}$. Stress path and deviator stress - axial strain curves obtained from CK_0UC and CK_0UE tests are depicted in Figure 32. Marked peaks of deviator stress were observed for the samples at deeper depths, but not for the sample at

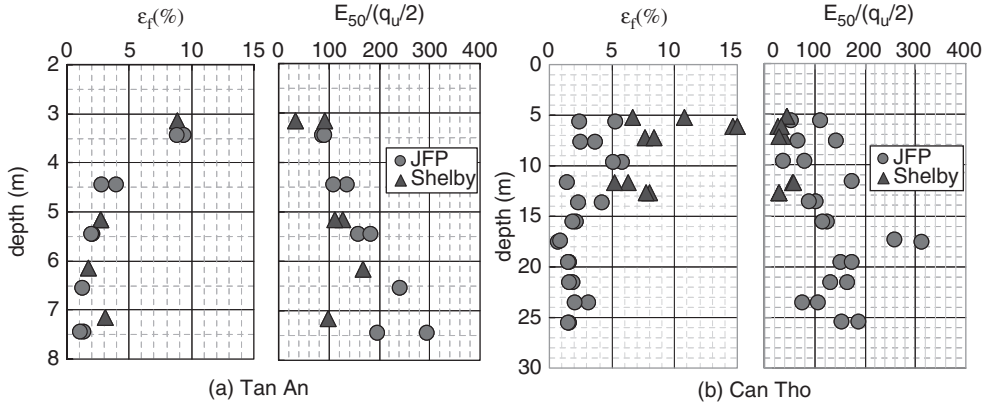


Figure 30. Sample quality in terms of ε_f and E_{50}/c_u from UC tests.

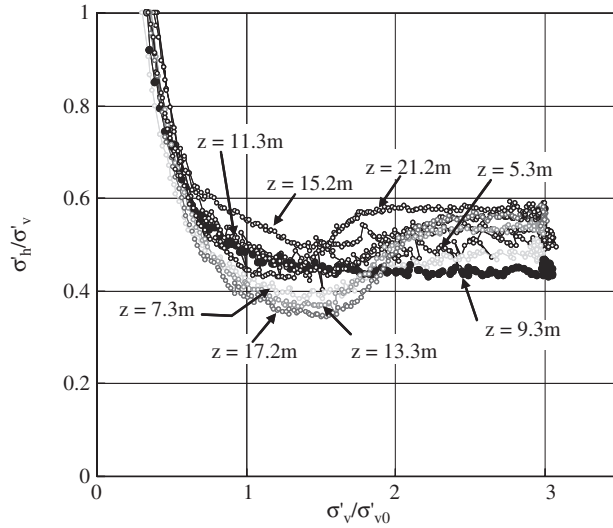


Figure 31. Relationship between σ'_h/σ'_v and σ'_v/σ'_{v0} observed in K_0 consolidation in triaxial test for Can Tho clay: JFP samples.

shallower depths especially $z = 9.6$ m. Strength anisotropy s_{u_CKoUE}/s_{u_CKoUC} are about 0.8 at the shallower depths ($z < 10$ m) and 0.6 at the deeper depth ($z > 11$ m).

6.4 Undrained strengths and sensitivity

For the Can Tho clay, field vane tests (FV) and laboratory vane tests (LV) were conducted to evaluate the undrained strength and sensitivity. The size of vane blade of FV is 80 mm in height and 40 mm in width and that of LV is 30 mm in height and 15 mm in width. Rotational rate of the vane tests were 6 degree/min for both FV and LV tests. The laboratory vane blade was inserted to the sample in the sampler to the depth of 85 mm. In Figure 33, the undrained shear strengths obtained from the FV and LV tests are plotted with the depth as well as the strength estimated from the other tests, UC tests, CK_0UC and CK_0UE tests, and constant volume direct shear test (DST). No correction was done for the undrained strength from the vane test according to plasticity index, such as the way proposed by Bjerrum (1972), because the plasticity indices of Can Tho clay are relatively low. The undrained shear strength from the FV and LV tests agreed well except at the depth of 9.3 m and 10.3 m where low plastic soils with thin sand seams deposit. For the shallower depth less than 12 m, the vane strength agreed well with the strength from DST and higher than $q_u/2$ from the JFP samples. It is interesting that the field vane shear

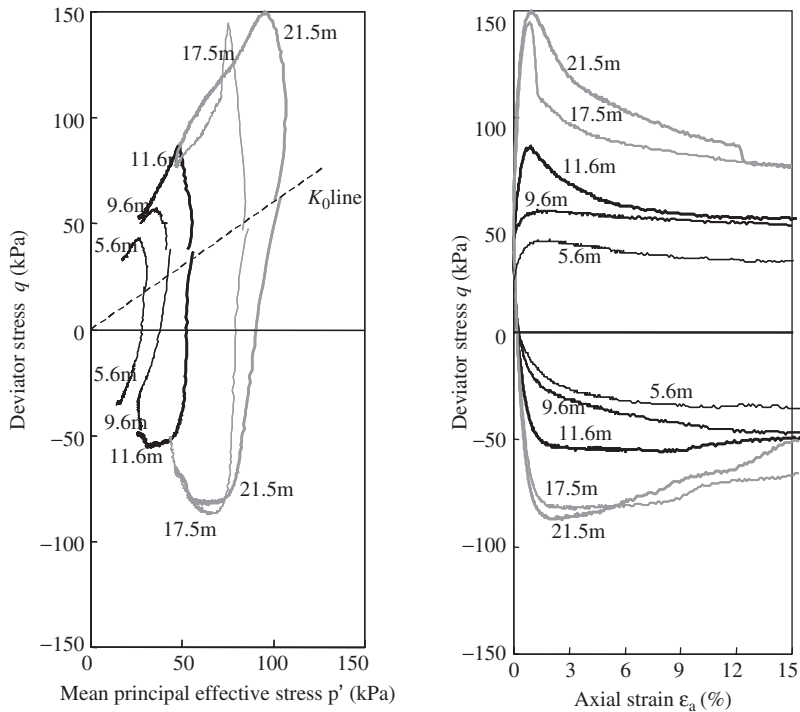


Figure 32. Results of triaxial CK₀U test for Can Tho clay: JFP samples.

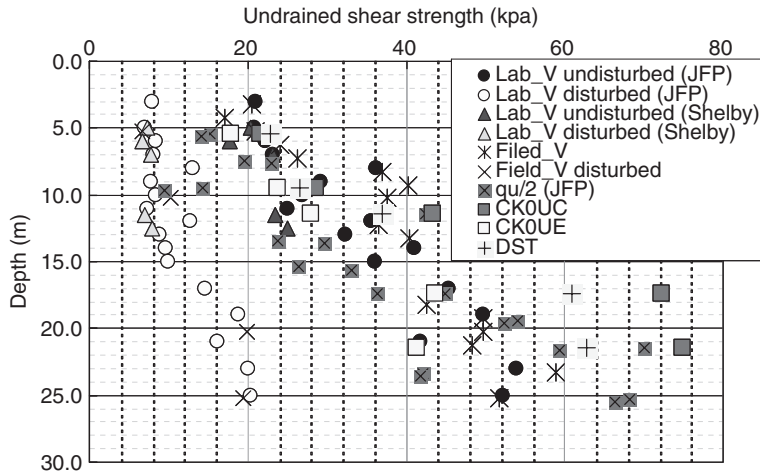


Figure 33. Undrained shear strength obtained from various tests for the Can Tho clay.

strength at the depth about 9.5 m is the largest among those obtained from various tests. For the deeper depth, however, the vane strengths are smaller than that from DST and almost equivalent to the $q_u/2$.

Sensitivity obtained from the field vane and laboratory vane tests are plotted with the depth in Figure 34. Depth variation of liquidity index, LI , is also shown in the figure. Sensitivity variation with depth at the Can Tho site shows similar trend of LI variation with depth. However, despite the large liquidity index greater than 1.0 and high degree of microstructuration discussed above, the sensitivity of Can Tho clay derived from the vane tests are relatively small ranging from 2.5 to 4.0.

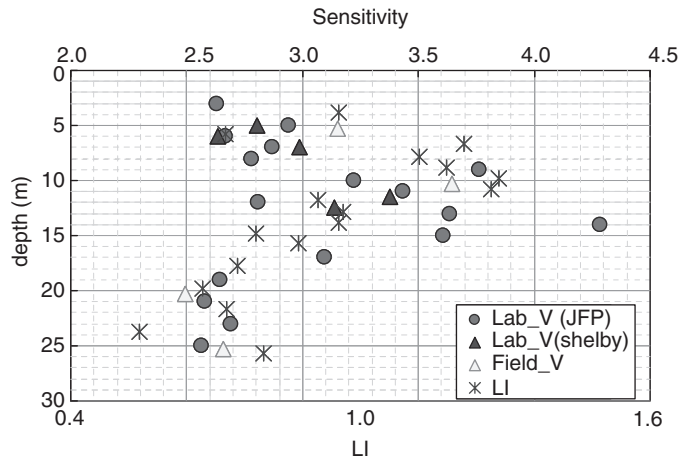


Figure 34. Depth Variation of sensitivity obtained from field and laboratory vane tests and liquidity index.

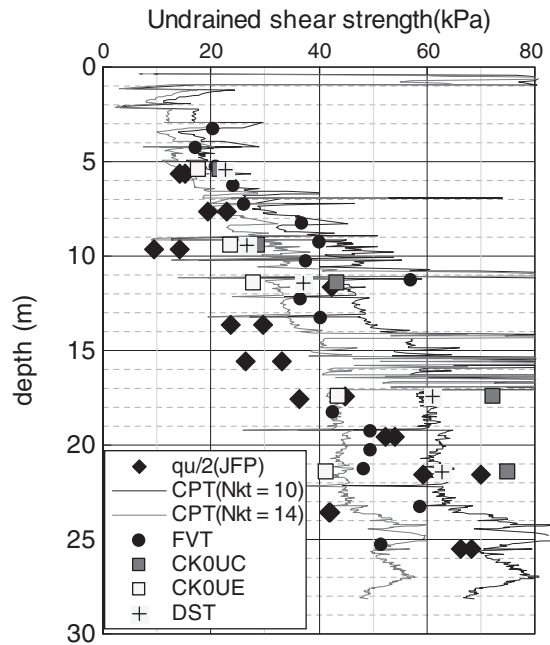


Figure 35. Comparisons of undrained shear strength obtained from CPT with those by the other tests.

6.5 Field tests

6.5.1 Piezocone cone penetration tests

For the soil profiles with large vertical heterogeneity like the Can Tho site, the cone penetration test has a strong advantage. The continuous measurement of quantities, such as the tip resistance and the pore water pressure behind the cone can capture the entire variation of soil profiles, some of which might be missed in the procedure of the sampling and laboratory testing of a certain intervals. However, interpretation of the cone test results is not straightforward. The undrained strength profiles estimated from the piezocone tests are given in Figure 35 with the strengths obtained from the other tests. The cone factor $N_{KT} = 10$ and 14 are assumed for evaluating the undrained shear strength. The field vane strength (s_{uFV}) at the shallower depth agrees well with the undrained strength from the CPT (s_{uCPT}) with

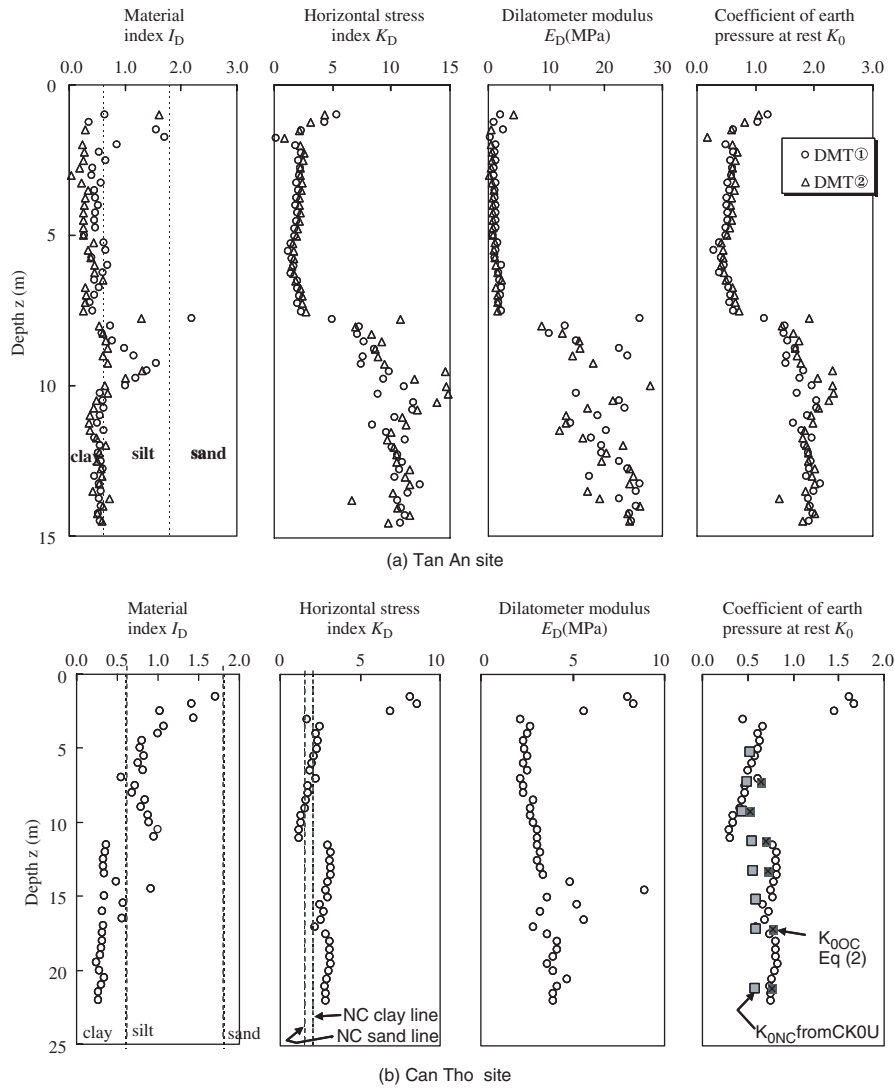


Figure 36. Results of dilatometer tests at Tan An and Can Tho sites.

$N_{KT} = 10$ but at the deeper depth the s_{uFV} becomes closer to the s_{uCPT} with $N_{KT} = 14$. The undrained shear strength of constant volume direct shear test (s_{uDST}), which is close to the average of undrained strength of CK_0UC and CK_0UE tests, has the opposite trend to the s_{uFV} in the relation to the s_{uCPT} . The possible reason of the difference is the effect of partial drainage which might take place during the vane tests at the shallower portion with low plastic soils and thin sand seams. The strength estimation of the low plastic soils, which might be categorized as intermediate soils, is a difficult issue, including the question if the undrained strength is a proper parameter in stability analyses. Application of cone test on this type of soils needs further studies, which is very crucial for the Mekong Delta clay with rapid deposition including coarser materials.

From Figure 35, it can be said that very good quality sample, like the ones retrieved at the deeper depths, is crucial for UC tests to evaluate a reasonable cone factor.

6.5.2 Flat dilatometer tests

Results of Marchetti's flat dilatometer test (DMT) done at Tan An site and Can Tho site are summarized in Figure 36. Overall trend of engineering properties at the two sites can be captured well by the DMT,

but very thin sand layer or seams at the depth of about 15 m at the Can Tho sites could not be detected by the DMT, although it indicates small increases of material index I_D and dilatometer modulus M_D . K_0 values calculated using equation (1), which is given by Marchetti (1980), are also shown in Figures 36.

$$K_{0DMT} = \left(\frac{K_D}{1.5} \right)^{0.47} - 0.6 \quad (1)$$

The K_{0DMT} values of the Tan An Clay are in the reasonable range from 0.45 to 0.7 for the depth of very soft high plastic and from the depth below 8 m, the values jump up to nearly 2.0. Due to the high stiffness, sampling could not be done at the depth below 8 m, however, it is inferred from the DMT results and the CPT results shown in Figure 7 that the soil in the stiff layer is hard slit or clay with very large OCR. There is a gap in the depth variation of K_{0DMT} of Can Tho clay at depth about 11.5 m. K_{0NC} obtained from the K_0 consolidation tests (Figure 31) and K_{0OC} calculated by the equation (2) proposed by Mayne and Kulhawy (1982) are shown in Figure 36(b) with the K_{0DMT} values of Can Tho clay.

$$K_{0OC} = K_{0NC} \cdot OCR^{\sin \phi'} \quad (2)$$

Although the gap of K_{0DMT} cannot be seen in the results of the triaxial K_0 consolidation tests, the K_0 values obtained from the two methods agree quite well. The K_{0DMT} values at the depths about 10 m are unrealistically small less than 0.3. For evaluating the K_0 value by using field tests, further studies are needed especially for the low plastic soils.

7 CONCLUSIONS

This paper has presented soil characterization studies done for the alluvial deposits at the Tan An and Can Tho sites in Mekong River Delta. The Tan An clay, which has been deposited at the riverside of meandering Bam Co Tay River, is highly plastic and very soft with relatively uniform physical properties to a depth of about 8 m. The Can Tho soft clay has been deposited at the flooding area beside the levee of Hau River, one of the major streams of Mekong River. It has significant heterogeneity in the depositional direction, having relatively low plasticity with thin sand seams, which might be created due to large floods. Depositional speed of the Can To clay is much faster than that of the Tan An clay. C_{14} dating of the Tan An clay at depth of 6.5 m and the Can Tho clay at depth of 19.5 m are almost the same, 6,300 BP. Although mechanical and physical properties of the two clays are quite different, both clays have developed high degree of microstructures, which has been confirmed from CRS tests. The ratios of maximum compression index just after the consolidation yield stress to that at the very large consolidation pressure are more than 6 for both clays if excellent quality of sample can be retrieved. For the non uniform and very thick soft deposit like the Can Tho clay, continuous soil sounding using piezocone is essential for characterizing the site.

Wide varieties of soft soils with different physical and chemical properties prevail in the huge area of Mekong River Delta. Continuous soil investigation studies and practices with good quality sampling, such as thin-walled tube sampler with fixed piston, and laboratory testing as well as appropriate field testing are highly recommended, which can greatly contribute to the proper development in Mekong River Delta.

ACKNOWLEDGEMENTS

The authors would like to acknowledge collaborators in the soil characterization studies, Dr. P. H. Giao (AIT), Mr. B. T. Man and Dr. N. H. Quan (HCMCUT), Mr. T. H. Hung (USCO), Mr. C. N. Sung and Mr. D. C. Thuan (TEDJ), Mr. K. Ohshiro and Mr. Y. Yuasa (Tokyo Tech), Mr. Y. Shiraishi (Nippon Koei Co., Ltd.), Dr. T. Fukasawa and Mr. H. Saegusa (Toa Corp.), and Mr. T. Ueda (Saeki Kensetsu Kogyo Co., Ltd.). The authors also are grateful to Taisei-Kajima-Nippon Steel Joint Operation for rendering the various supports during the field investigation at Can Tho bridge construction site. The authors gratefully acknowledge the funding for this research from Japan Society for the Promotion of Science (JSPS) as Grant-in-Aid program.

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Geotechnical properties of Hachirogata Clay

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ABSTRACT: A series of site investigations was carried out at the site of Hachirogata, where Holocene Clay is thickly deposited. The distinguishing feature of Hachirogata Clay is high index properties; for example, its plasticity index exceeds 100. The reason for such large index properties may be attributed to the presence of clay minerals with high activity and large organic content. In addition, another possible reason is the presence of a large proportion of microfossils, especially diatoms, which have been observed by Scanning Electron Microscope (SEM). The representative size of these diatoms is about $10\text{ }\mu\text{m}$, which is classified into silt size. However, diatoms have a large capacity for holding water, because of their porous structure. This paper deals with interesting properties of Hachirogata Clay, compared with other Japanese soils and overseas soils based on the data accumulated by the author through his site investigations.

1 INTRODUCTION

The Hachirogata site is located in Akita prefecture, in the northern part of Honshu Island, Japan. Hachirogata was the second largest lake in Japan before its reclamation in the 1970's as cultivated land for a rice paddy. The investigated site is located at the center of the reclaimed lake, where the soft clay is thickly deposited to more than 45 m. PARI (Port and Airport Research Institute, formally PHRI, Port and Harbor Research Institute) carried out a series of site investigations to study geotechnical properties of the soft clay, especially sampling quality retrieved by several types of samplers, such as the Japanese standard sampler (fixed piston thin wall sampler), Shelby tube sampler and Laval sampler (Oka et al. 1996).

The distinguishing feature in the geotechnical properties of Hachirogata Clay, compared with those of soft clays at other sites, is its high index properties; for example, its plasticity index (I_p) exceeds 100. The author has not previously experienced a clay with I_p more than 100, although I_p for Japanese marine clays is generally large, compared with that of other overseas clays. Even in the literature, studies on clay with such high I_p can be rarely found except for Mexico clay, which is well known for large settlements in the foundation of high-rise buildings in Mexico City (see, for example, Mesri et al. 1975; Dias-Rodriguez 2003).

It is found from observation by Scanning Electron Microscope (SEM) that Hachirogata Clay contains a large proportion of microfossils, especially diatoms. It is anticipated that if soil contains diatoms, its index properties as well as its natural water content can become very large, because of large pores within diatom particles (Tanaka & Locat 1999; Locat & Tanaka 2001; Shiwakoti et al. 2002). This paper presents geotechnical properties of Hachirogata Clay with high I_p and compares them with those having moderate I_p .

2 GEOLOGICAL PROCESS OF HACHIROGATA LAKE

Hachirogata is located about 30 km north of Akita city, facing the Japan Sea, as shown in Fig. 1. The Oga peninsula, which is located in the west side of the site, used to be an isolated island created by volcanic activities in the late Pleistocene era. This island was connected by sand banks in the west and south sides to the main land, prior to the Holocene era. The area surrounded by the sandbanks and the Oga island became the lake called "Hachiro-gata" ("gata" means lagoon in Japanese), becoming the second

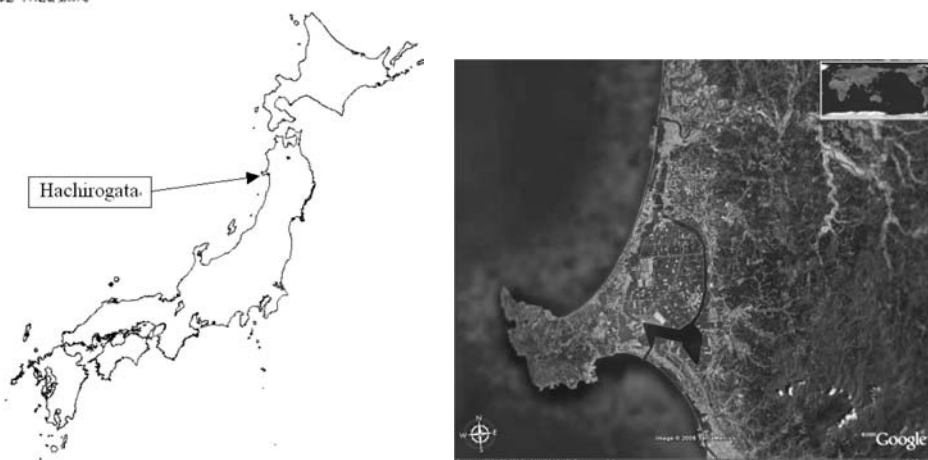


Figure 1. Location of site investigation for Hachirogata Clay.

largest lake, after Biwa Lake in Japan, before the lake was finally reclaimed. The clay layer investigated in this report was deposited under blackish or lacustrine environmental conditions, since sea water was coming through only a restricted narrow channel in the south sandbank. The lake prior to reclamation was famous for plenty of brackish fish. The water depth of the Hachirogata Lake was relatively shallow and ranged from 4 to 5 m. The work of reclamation was started in 1957 by constructing dikes, whose length is 52 km in total along the shore of the original Hachirogata Lake. It took as long as 20 years for reclamation and the work was completed in 1977 (more information on the geological process may be referred to Shiraishi 1990).

The thickness of clay layer varies with locations, being deepest at the center and gradually becoming thinner towards the fringe of the lake. The investigated site is located nearly at the center point, where the thickness of the soft clay is as much as 45 m.

3 MAIN PHYSICAL AND CHEMICAL PROPERTIES

Figure 2 shows typical physical and chemical properties of the Hachirogata Clay. The soil samples were recovered from as deep as 45 m. Below a depth of 45 m, a stiff sand layer was found. A soil profile measured by Piezocone Penetration Test (CPTU) is shown in Fig. 3, where q_t , u and f_s are the point resistance considering the effective area ratio of the cone, the pore water pressure and the skin friction, respectively.

As shown in the profile obtained by CPTU, the layer is very homogeneous and any high permeable layers are not found. The original ground surface is covered by sandy soil to 3 m depth. As shown in grain composition, the soil largely consists of silt and clay particles (where the boundary particle size is $2\ \mu\text{m}$). The grain composition of the upper layer down to 8 m depth is mainly silt, then the clay content gradually increases with depth and the clay content is the largest at about 30 m depth. On the other hand, index properties such as liquid limit (w_L) and plastic limit (w_P) decrease with depth in spite of an increase in the clay content. The natural water content (w_n) is nearly equal to w_L down to a depth of 26 m, in another word, the liquidity Index (I_L) is 1.0. However, w_n becomes smaller than w_L below a depth of 30 m, where w_L again increases with depth.

Deposition age for the sediments was measured by isotope of ^{14}C and it was found that the depositional age at 5, 19 and 35 m depths are 3500, 6300 and 8000 year Before Physics (BP), respectively. These dates indicate that the investigated soil layer in this study was deposited in the Holocene era, i.e., after the ice age. Considering these deposited ages and depths of the measured age, it is estimated that the rate of deposition gradually decreased with time: 10 mm/year (8000 BP–6300 BP); 5 mm/year (6300 BP–3500 BP); 1.6 mm/year (3500 BP–the present). The speed for depositing these layers is considerably faster compared with Holocene deposits in other areas in Japan.

Salinity of liquid in the pores of the soil is shown in the right end of the column of Fig. 2. The salinity, being presented by content of chlorite to the dry weight of soil (CH), slightly varies with depth and the

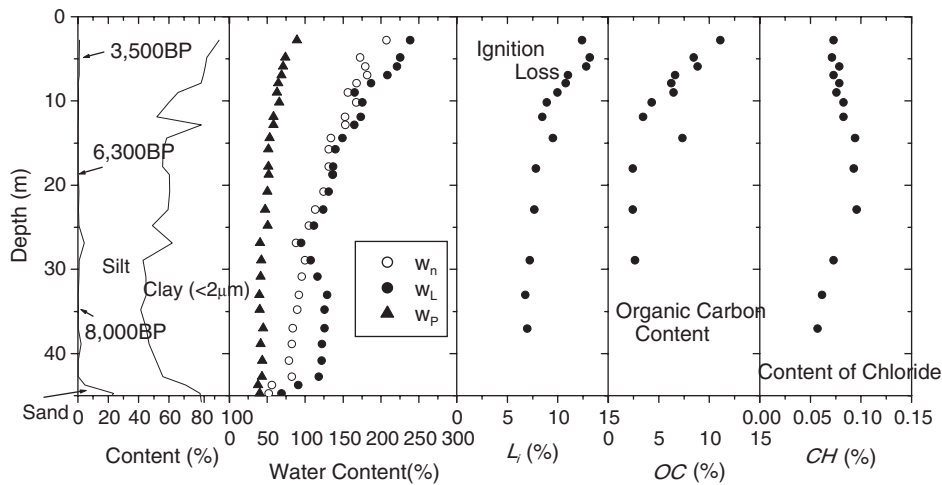


Figure 2. Physical and chemical properties at the site of Hachirogata.

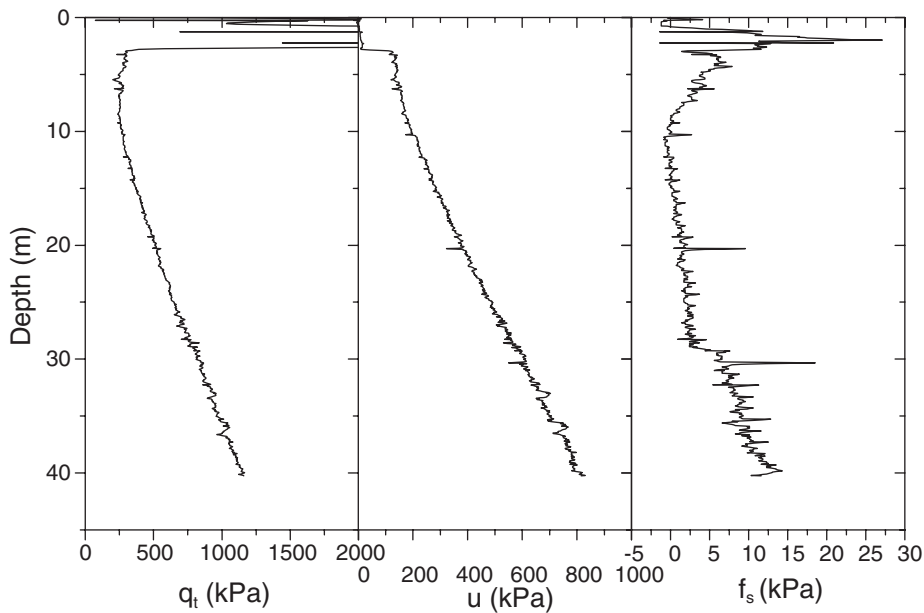


Figure 3. Soil profile measured by Cone Penetration Test.

largest CH is measured at the middle point of the layer. This depth corresponds quite well with the depth where the I_L becomes suddenly smaller (w_n becomes smaller than w_L). It is inferred that at this time, the depositional environment may have changed probably due to creation of the sandbank which separated the lake from the sea. In general, the relatively small CH compared with sea water indicates that the environmental deposition in the soil layer may have been brackish, which is consistent with the diatom species observed by Scanning Electronic Microscope (SEM), as will be discussed later.

Index properties for Hachirogata Clay are extremely large through all depths, especially near the ground surface. The activity for various soils investigated by the author's group is indicated in Fig. 4. In this figure, clay contents (the diameter of soil particles is less than $2\mu\text{m}$) are plotted on the horizontal axis and I_p on the vertical axis, so that the slope in the two values presents the activity. Although it is well known that the activity (A_c) for Japanese clays is, in general, much larger than that for other overseas

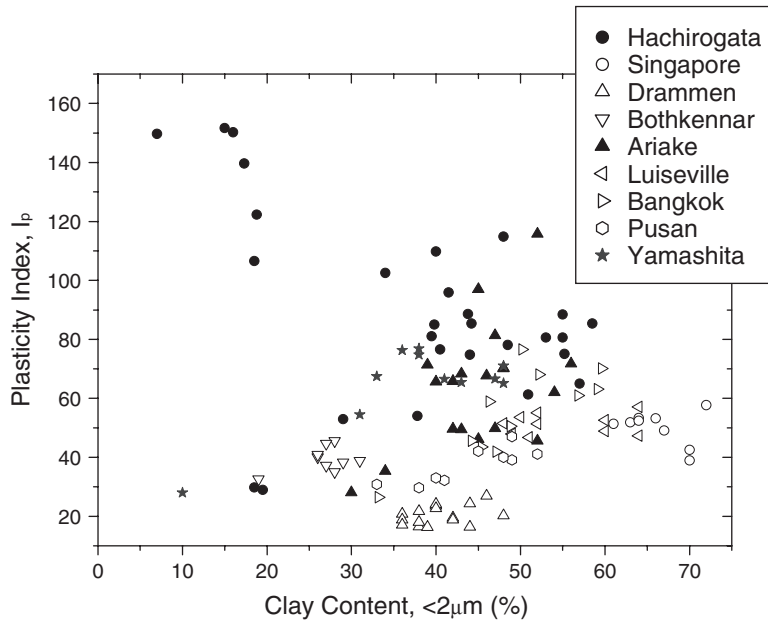


Figure 4. Activity of various soils in the world.

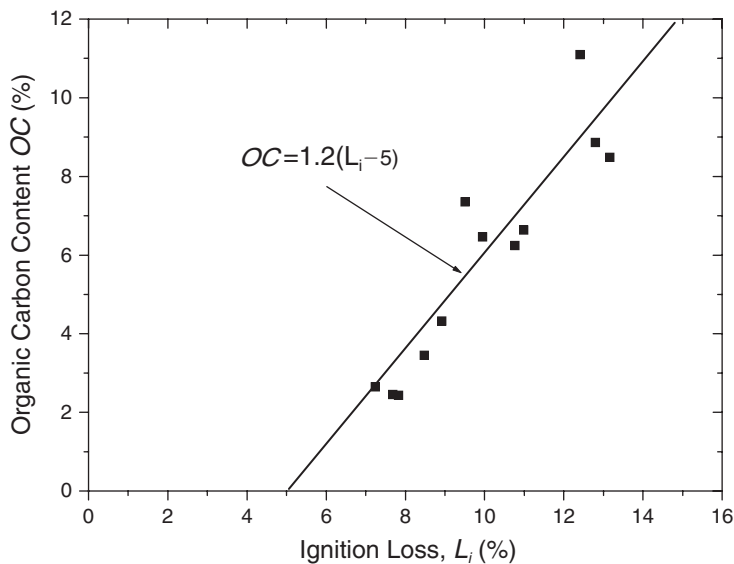


Figure 5. Relation between L_i and OC .

clays, the activity for Hachirogata Clay is extremely large, especially at shallow depths (A_c is more than 5). When depths are deeper than 15 m, their activity becomes close to that for other Japanese clays (in the figure, Ariake and Yamashita are Japanese clays).

The organic matter content was investigated by two types of tests: ignition loss (L_i) and organic carbon content (OC), following the Japanese Industrial Standard (JIS) or the Japanese Geotechnical Society (JGS) Standards (JIS A1226:2000 for L_i and JGS0231-2000 for OC).

There exists a clear correlation between L_i and OC , as shown in Fig. 5. When L_i exceeds 5%, OC increases with the increase in L_i . This relation suggests that 5% of L_i is caused by loss not attributed to the organic matter, i.e., dehydration of clay minerals in the form of absorbed water or water of hydration

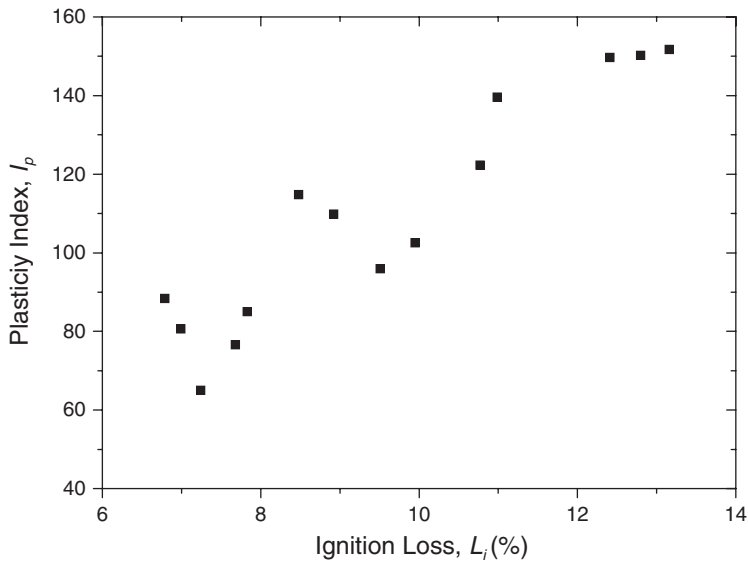


Figure 6. Relation between I_p and L_i

due to high temperature of 750°C used in the test to determine OC . Also I_p can be correlated with L_i as shown in Fig. 6. Large index properties for Hachirogata Clay may be attributed to the large content of organic matter. This point will be discussed later in more detail.

Figure 7 shows X-ray diffraction charts for Hachirogata Clays at four different depths. The tested grain size of all specimens was less than 2 μm . The specimens were prepared by three different treatment methods: natural (denoted by N in the figure); Glycolation (G) treated; heated at 550°C (H). It can be seen that the Hachirogata Clay might contain a lot of smectite, whose activity is very high. Locat et al. (1996) have also shown that main clay mineral in Japanese clays is smectite. The large index properties and high activity of Hachirogata Clay may be attributed to the presence of a large amount of smectite. It is reported, however, that the ϕ' values for pure smectite are smaller compared to that of illite or kaolin whose I_p are relatively small (see for example, Mitchell 1976). However, as described later in more detail, ϕ' of Hachirogata Clay is large, compared with other soils having low I_p .

4 INFLUENCE OF MICROFOSSILES, ESPECIALLY DIATOMS

Figure 8 shows the microfabric of Hachirogata Clay at a depth of 6 m, observed by Scanning Electronic Microscope (SEM). Large numbers of diatom skeletons with the shape of a hollow cylinder can be seen in the figure: they are diatoms. It is anticipated that if a soil contains a lot of diatoms, the large sized pores in their skeletons lead to high water content. It is seen from the figure that the size of the diatoms is almost 10 μm , being the size of silt. It has been believed that due to electric charges on the surface of clay particles, a volume of water is attracted by the clay particles, and this amount of water is different for different clay minerals. If soil contains diatoms, however, a large part of the water held is entrapped in the pores of their skeletons, which may be classified as silt by the grading test. Therefore, it may be inferred that the Atterberg limits are strongly influenced by the presence of diatoms. In addition, diatoms may affect mechanical properties such as ϕ' value, because of their rough and angular shape as shown in Fig. 8.

The influence of diatoms on the physical, hydraulic as well as mechanical properties have been studied by artificially mixing ordinary soil with diatomite soil by, for example, Tanaka (2002); Shiwakoti et al. (2002). It was found from these studies that with the increase in content of diatoms, both w_L and I_p increase even though the clay content decreases. These behaviors are completely different from those of sandy or silty soils in which, with the decrease in clay content, w_L and I_p also decrease. Regarding mechanical properties for such diatomite mixtures, ϕ' and s_u/p increase with an increase in diatomite content. It should also be noted that solid particles density (ρ_s) of diatoms is very small compared to that of ordinary soil, around 2.7.

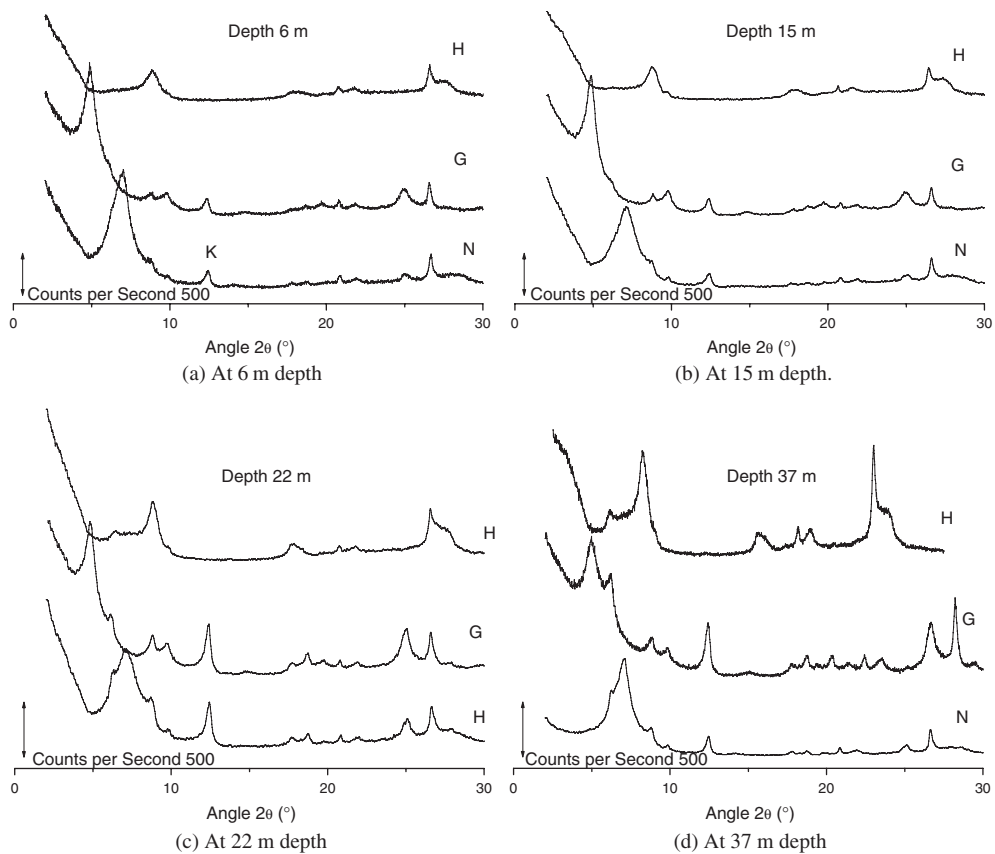


Figure 7. Clay minerals measured by X-ray diffraction test. N: natural, G: treated by Glycolation, H: heated at 550°C

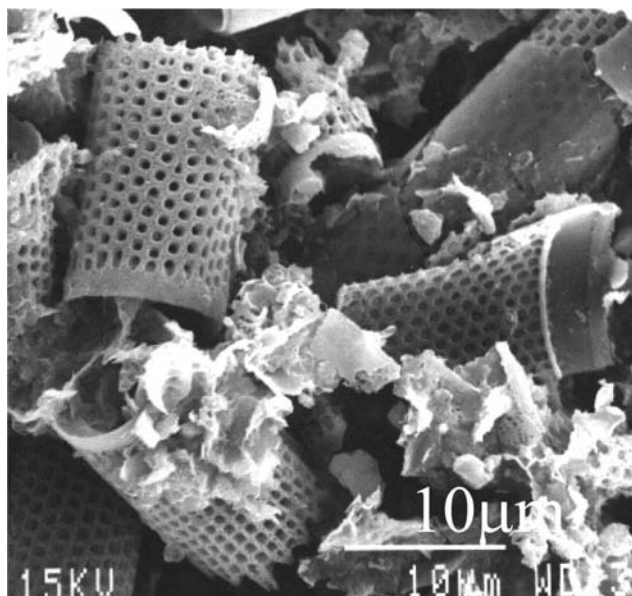


Figure 8. Microfabric of Hachirogata Clay using scanning electron microscope. Sample depth is 6 m.

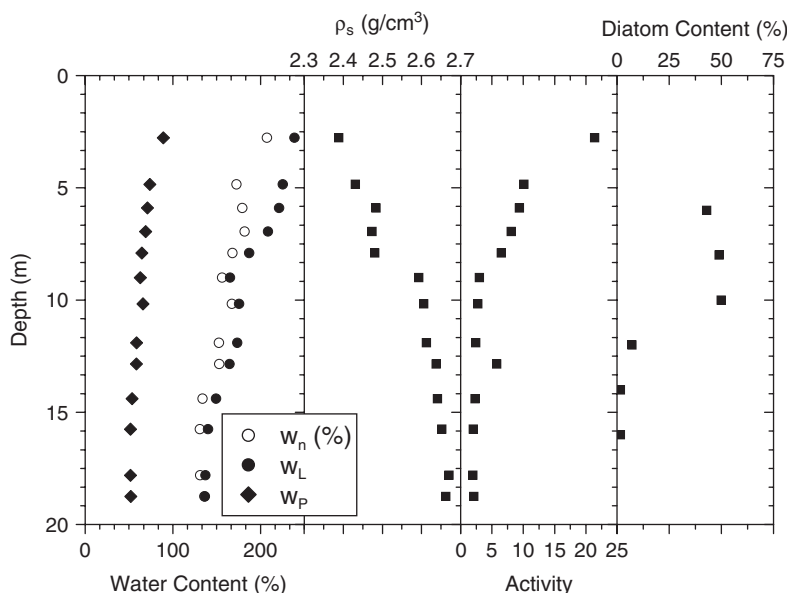


Figure 9. Influence of diatom contents on index properties, solid particles density and activity.

Table 1. Comparison of Index properties of Hachirogata Clay before and after consolidation of 10 MPa. Sampling depth is 6 m depth.

	w_L (%)	w_P (%)	I_p
Before (intact)	243.9	67.0	176.9
After (10 MPa)	179.8	55.5	124.3

The ingredient of diatoms is mostly silica (SiO_2), which is sometimes called biogenic or amorphous silica because of its non-crystalline structure. There are still controversies on how to quantify biogenic silica, since some parts of non-biogenic silica derived from quartz etc., is also inevitably measured in the process of quantifying total quantity of biogenic silica (see, Kamatani & Oku 1999). A preliminary method has been developed for analyzing the amount of diatoms by Shiwakoti et al. (1999). A specimen, weighing 0.6 g by dry weight, was treated with hydrogen peroxide and hydrochloric acid to remove organic matter and disperse diatom cells from soil particles. Part of the solution was taken with a pipette and mounted on a glass sheet. The specimen, thus prepared, was observed by optical microscope under 400 times magnification. The numbers of diatoms in a specified area were counted, from which the total number of diatoms present per unit dry weight of soil was calculated. Of course, there are many problems in evaluation of diatom numbers by this method. The size of diatom is not the same but varies with species. Also some diatoms are not in the original shape, but are found broken into smaller pieces. In this test procedure, such a fragment is counted as one. Therefore, the number of diatoms determined by this method does not precisely indicate the volume of diatoms present in the soil specimen, nonetheless, it provides at least a rough idea on whether a soil contains plenty of diatoms or not.

Based on knowledge of the influence of diatom presence from artificial mixtures of diatomite soils, the physical properties of Hachirogata Clay were examined again. It can be seen in Fig. 9 that at shallow depths, ρ_s is very small, whereas its activity is extremely large. It is very interesting in that at these depths, diatom content is very large. From this evidence it may be concluded that such large index properties are partly due to diatoms, in addition to two important facts: the clay mineral is smectite; and high content of organic matter cannot be ignored for large index properties.

There is further indirect evidence of the existence of diatoms in the soil near the ground surface. Table 1 compares the Atterberg limits before and after consolidation at 10 MPa. The sample was recovered from

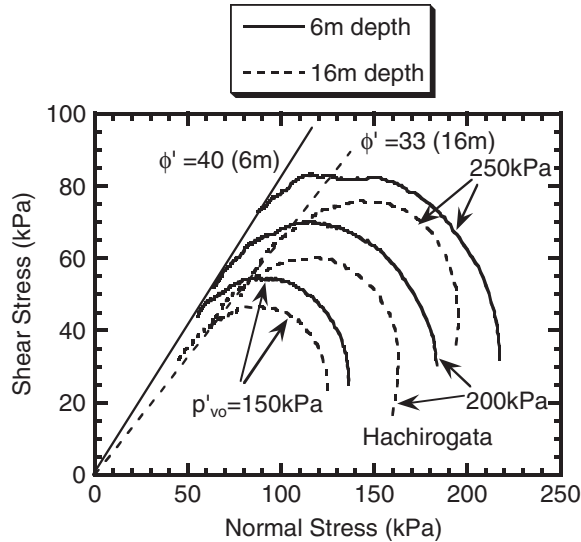


Figure 10. Test result from direct shear test. A sample of 6 m depth contains a lot of diatom compared with that of 16 m depth.

a depth of 6 m. The w_L for a consolidated specimen decreases from 244% to 180% because the skeleton has been crushed by the large consolidation pressure, so that the same quantity of water cannot be held in its pores. Other evidence is the strength. Figure 10 shows a test result from the direct shear test for specimens at depths of 6 m and 16 m. In this test, the specimen was sheared under constant volume, as will be described in 6.1. From Fig. 9, it is known that a specimen at a depth of 16 m contains a smaller amount of diatoms compared to that at 6 m. The value of ϕ' for a specimen at 6 m depth (40°) is much higher than those at 16 m (33°). Also the peak undrained strength normalized by the p'_{vo} at 6 m is greater than that at 16 m. This test result implies that the existence of diatoms creates higher ϕ' and the undrained shear strength, which is the same result as that obtained in the artificially prepared diatomite mixture (see Tanaka 2002; Shiwakoti et al. 2002).

5 CONSOLIDATION PROPERTIES

5.1 e - $\log p$ relation

Figure 11 shows e - $\log p$ relations measured by Constant Rate of Strain (CRS) tests, where a constant strain rate of $0.02\%/min$ ($3.3 \times 10^{-6}s^{-1}$) was applied. In this test, the specimens were recovered by two different samplers: the Japanese standard sampler with a stationary piston (JPN in the figure) and Shelby tube without a piston. As revealed by Tanaka (2000), sample quality retrieved by the Shelby tube is considerably poorer compared with that by the Japanese sampler, evaluated by the unconfined compression strength (q_u). It is apparent in Fig. 11 that the e - $\log p$ relations for the JPN sampler are located above those for the Shelby tube. However, this difference is caused by sampling depth (the Shelby tube was used at depths deeper than 12.8 m). Figure 12 compares the e - $\log p$ relations at nearly the same depth for JPN and Shelby tube samplers. It may be concluded that the e - $\log p$ relation is not much influenced by sampling methods: i.e., both samplers present the similar e - $\log p$ relation, which indicates flat relation (low compressibility) before the yield consolidation pressure (p_c) and sharp reduction of e after p_c . As will be mentioned later, however, the q_u value is strongly affected by sampling method. This apparent inconsistency is also observed in other sites, for example, Ariake, where there was no notable difference in the e - $\log p$ relations for different samplers including JPN and Shelby tube samplers, though there was a great difference in the q_u values. However, for Bothkennar clay, poor quality samples provide small q_u values, and the e - $\log p$ relations clearly are different from high quality samples (see Tanaka 2000 for more details).

The p_c values are shown in Fig. 13, compared with the in situ effective overburden pressure (p'_{vo}). It is well known that even in mechanically normally consolidated layers where the soil has not experienced a larger consolidation pressure than the present p'_{vo} , the p_c value is somewhat greater than p'_{vo} . Bjerrum

(1973) insisted that such an apparent overconsolidation is caused by ageing and called such clay “aged normally consolidated”. For Hachirogata Clay, the apparent overconsolidation ratio (OCR) is rather small, especially near the ground surface, but with the increase in depth, the OCR value increases to a value of about 1.2. It is again confirmed in the figure that the p_c value is not influenced by sampling methods. Figure 11 shows the e - $\log p$ relations at various depths. As consolidation pressure increases, for example, more than 200 kPa, it seems that the e - $\log p$ relations concentrate into a single relation, except for the specimen at a depth of 2.8 m. It is convenient to use relative void ratio for making a comparison of the e - $\log p$ relations at different depths because index properties for these samples also vary with depth. For such a purpose, several relative void ratios have been proposed, for example,

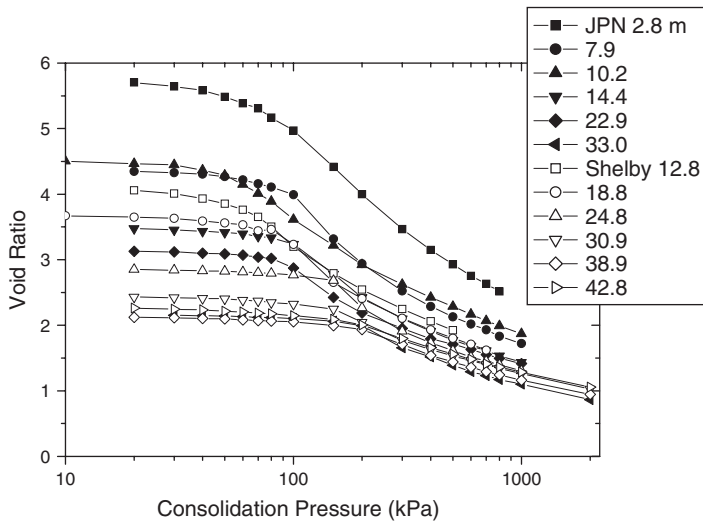


Figure 11. e - $\log p$ relation measured by CRS test with $3.3 \times 10^{-6} \text{ s}^{-1}$ strain rate.

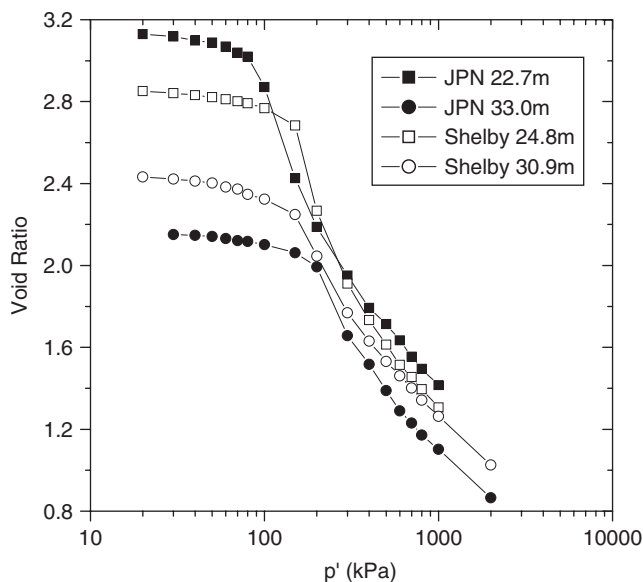


Figure 12. Selected e - $\log p$ relation. Samples were recovered by Japanese Standard sampler (JPN) and Shelby tube.

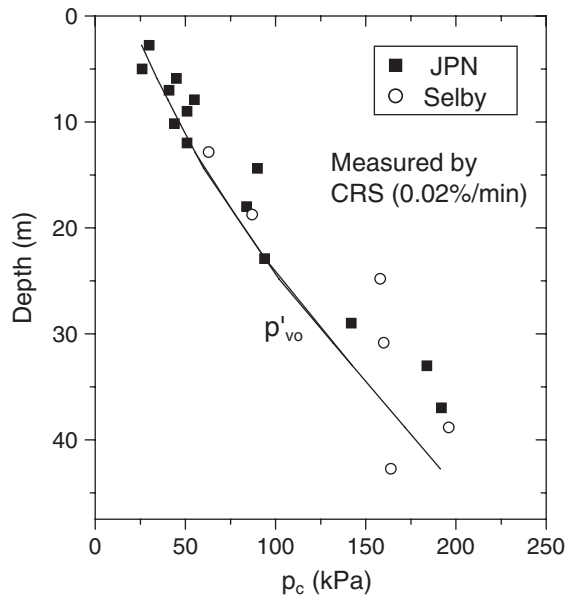


Figure 13. Yield consolidation pressure for Hachirogata Clay.

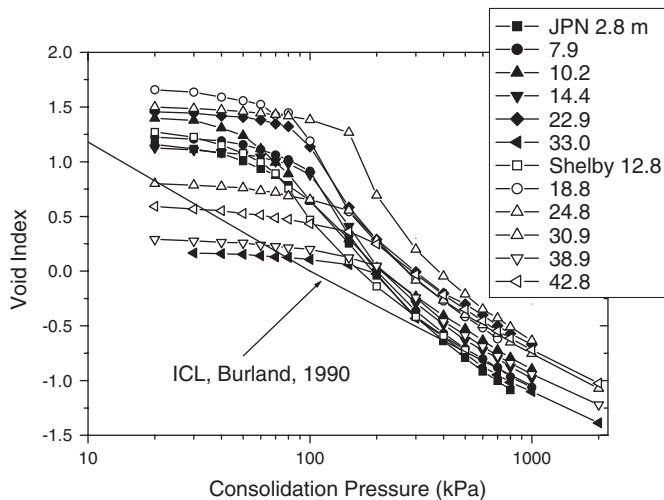


Figure 14. Relation with void index (I_v) and consolidation pressure.

Burland (1990); Tsuchida (2001), etc. The liquidity index I_L is one of the relative void ratios. Figure 14 shows the e – $\log p$ relation using Void Index (I_v), which was defined by Burland (1990) and calculated using w_L . According to Burland's concept, as p increases and soil structure is destructed, then the I_v – $\log p$ relation will be approaching a single line named "Intrinsic Consolidation Line" (ICL). It can be seen in the figure, however, that most I_v s at these pressures are greater than those along the ICL, although the slope of I_v – $\log p$ is almost the same as that of ICL. In the relation between I_v and $\log p$, samples obtained from shallow depths should be started from large I_v because of small p'_{vo} . However, as seen in Fig. 14, the I_v value at the starting point for shallower depths is not always larger than that for samples collected from great depths.

Figure 15 shows the relation between I_{vo} and p'_{vo} , where I_{vo} is the in situ void index, calculated using initial void ratio. In this figure, the relation between I_{vo} and p'_{vo} for other various sites are also plotted, in addition to data from Hachirogata. Due to the fabric structure created in the process of sedimentation, it

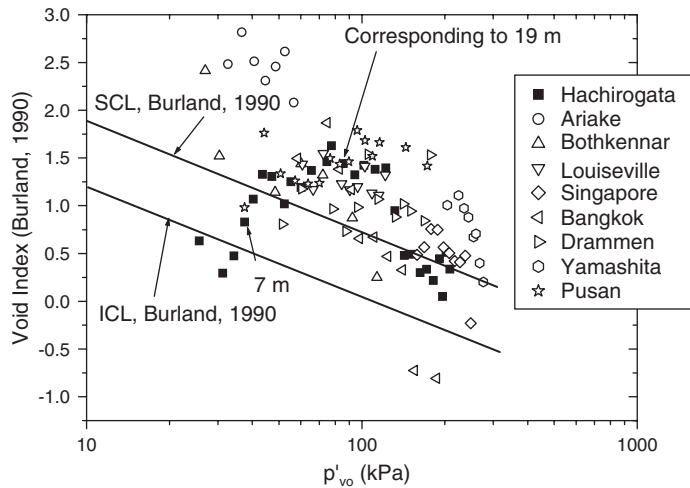


Figure 15. Relation with initial void index (I_{v0}) and in situ effective overburden pressure (p'_{v0}) for Hachirogata Clay as well as other clays from the author's investigation.

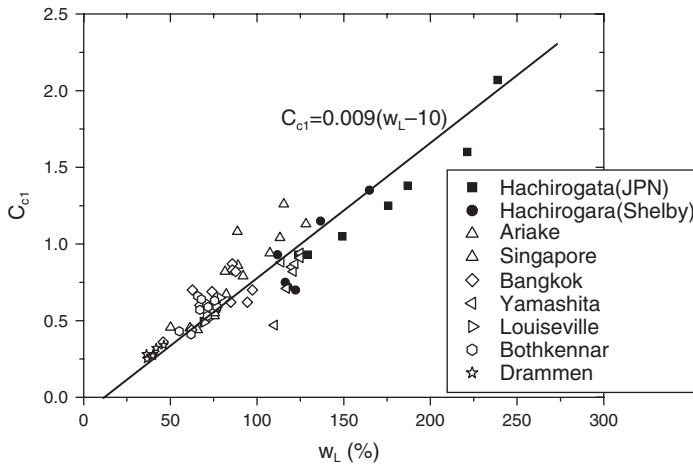


Figure 16. Relation with w_L and C_{c1} , which is compression index at consolidation pressure large enough to be constant.

is known that I_{v0} is somewhat larger than that along ICL. Burland (1990) accumulated data of the I_v -log p'_{v0} relations for naturally deposited ground worldwide to draw a line of the relation between I_{v0} and log p'_{v0} . He named this line "Sedimentation Compression Line" (SCL). For all soils except for Hachirogata Clay, as p'_{v0} increases, I_{v0} decreases because of an increase in consolidation pressure. For Hachirogata Clay, however, I_{v0} increases with increase in p'_{v0} and reaches a peak at about p'_{v0} of 80 kPa (this pressure corresponds to 19 m depth), then I_{v0} decreases with increasing p'_{v0} . The I_{v0} at small p'_{v0} is lower than even that of ICL. This strange behavior may be due to high content of organic matter or microfossils, especially diatoms, as described before. When depth is greater than 19 m, the relation between I_{v0} and p'_{v0} joins the range of normal soils. This fact indicates that below a depth of 19 m, the influence of organic matter as well as diatoms is not so strong as to behave oddly.

5.2 Compression index

Compression index (C_c) has been correlated with index properties, especially w_L for many years. When it is attempted to establish such a correlation, it should be noted that C_c is not a constant even in the

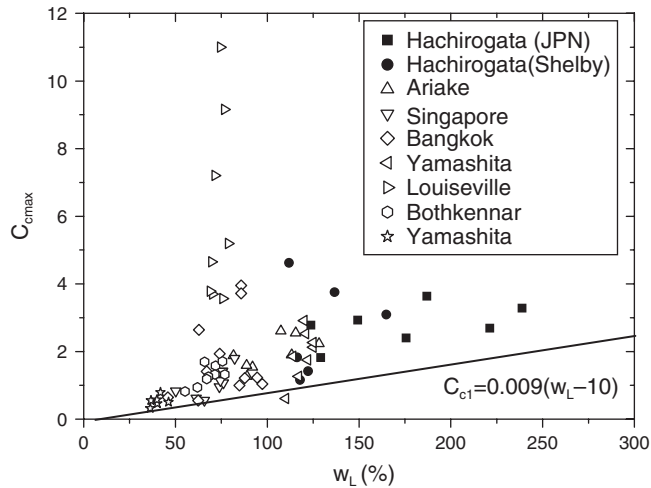


Figure 17. Relation with w_L and C_{cmax} , which is the maximum of C_c in the e - $\log p$ relation.

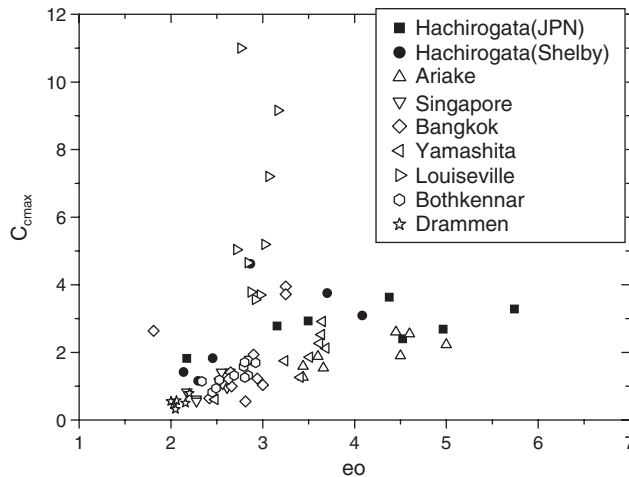


Figure 18. Relation with initial void ratio (e_o) and C_{cmax} , which is the maximum of C_c in the e - $\log p$ relation.

normally consolidated (NC) state as shown in Fig. 11, for example. When p exceeds the p_c value by more than about 10 times, a constant C_c is usually obtained. If the C_c under these pressures is denoted by C_{c1} , C_{c1} can be strongly correlated with w_L , as shown in Fig. 16. The relation of C_{c1} with w_L follows the relation proposed by Terzaghi: $C_{c1} = 0.009(w_L - 10)$, including other soils accumulated by the Author's investigations at various sites.

On the other hand, the steepest gradient of the e - $\log p$ relation (C_{cmax}) cannot be correlated with w_L (see Fig. 17). C_{cmax} is not related to the fundamental properties such as w_L , but probably has a strong relation with structure. C_{cmax} is plotted against initial void ratio (e_o) in Fig. 18. Except for Louisville clay, the relation with e_o and C_{cmax} is distributed in the some range, including Hachirogata Clay.

5.3 Hydraulic conductivity

For calculation of settlement, the rate as well as magnitude of settlement is important. It is known that hydraulic conductivity (k) reduces with decrease in void ratio, and its reduction is expressed by $\Delta \log k = a \Delta e$, where a is a constant. Figure 19 shows the relation between e and $\log k$ for Hachirogata

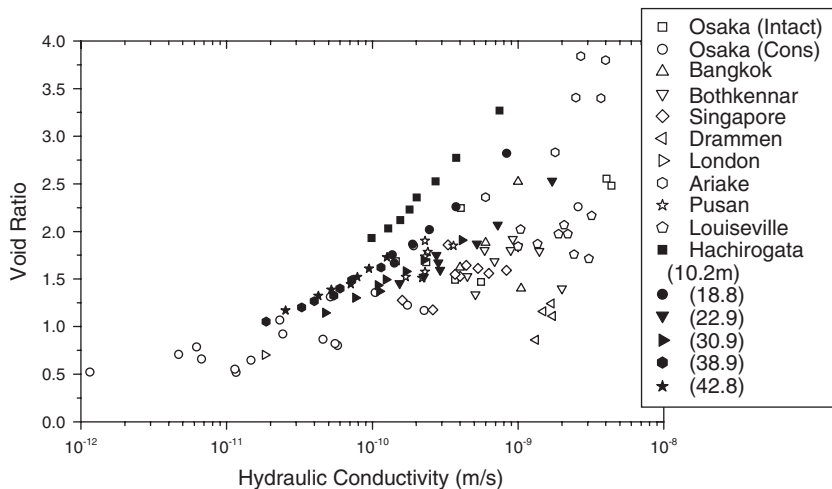


Figure 19. Comparison of hydraulic conductivity for Hachirogata Clay with various soils.

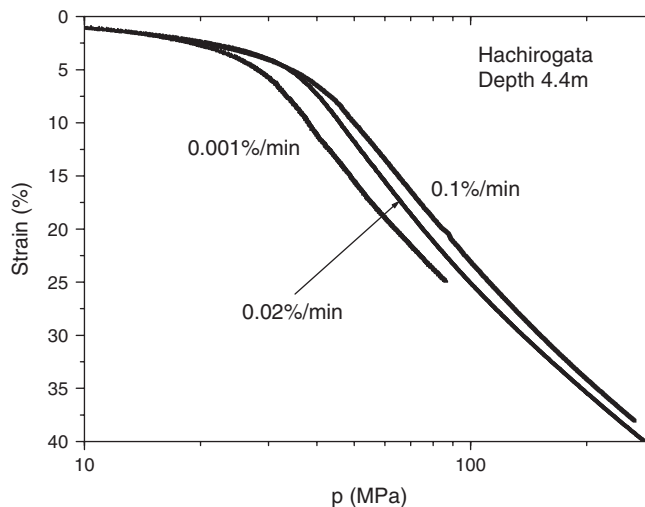


Figure 20. e - $\log p$ relation with different strain rate.

Clay measured by CRS test, in comparison with other clays. It can be seen that the location of e and $\log k$ relation for Hachirogata Clay is in the upper band of the figure: i.e., Hachirogata Clay is rather impermeable in spite of large e . This may be explained by the fact that, despite the abundance of diatom skeletons having large porous structures, continuity of pores is rather poor, apparently due to the considerable presence of a poorly permeable mineral like smectite.

5.4 Rate effect

It is known that since clay is a viscous material, the e - $\log p$ relation is influenced by the strain rate in the same manner as strength. Figure 20 shows the e - $\log p$ curves measured by CRS tests under different strain rates. As can be seen, when the strain rate is larger, the e - $\log p$ relation is shifted to the right. This amount of shift is nearly proportional to the increase in the strain rate in logarithm scale. The ratio of the p_c value due to an increase in the strain rate by 10 times is indicated in Fig. 21, where the range of strain rate for the experiment is from 0.001%/min and 0.01%/min. Tanaka et al. (2000) tried to find out

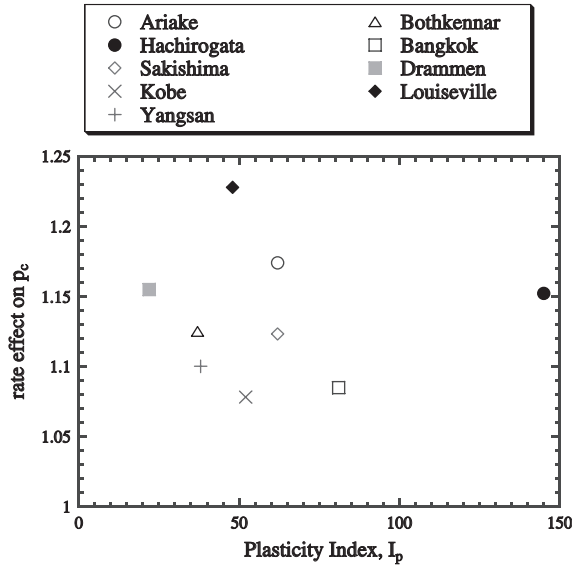


Figure 21. Rate effect on the p_c value for various soils (after Tanaka, et al., 2000).

factors influencing the change in the p_c value. However, there is no single factor governing the strain rate effect on the e - $\log p$ relation. The only clear finding from their study was that the change in the p_c value was not dependent on I_p , as shown in Fig. 21.

6 STRENGTH PROPERTIES

6.1 Undrained shear strength

Undrained shear strength (s_u) was measured by Field and Laboratory vane, unconfined compression and direct shear tests. The laboratory vane test was carried out in the following manner. First, the soil sample was extruded about 7 cm from the sampling tube, for index tests such as Atterberg limit tests. Then, a vane with 20 mm in diameter and 40 mm high was inserted into the sample in the sampling tube (its diameter was 75 mm). The rotation speed of the vane was $6^\circ/\text{min.}$, following the standard of JGS (see Tanaka 1994 for more detail). The size of specimens for the direct shear tests was 60 mm in diameter and 20 mm in height. Specimens were reconsolidated to p'_{v0} , then sheared at a shearing speed of 0.25 mm/min under constant volume (equivalent to the undrained condition) (see Takada 1993). Unconfined compression tests followed the standard of the JGS: i.e., the diameter and height of a specimen were 35 mm and 80 mm, respectively. The axial strain rate was 1%/min.

The s_u values measured from these tests are plotted in Fig. 22. Laboratory tests (Unconfined compression, laboratory vane and direct shear tests) were carried out for samples with two different qualities, i.e., retrieved by the Japanese standard sampler and the Shelby tube. From the unconfined compression test, the value of s_u ($= q_u/2$) for samples collected by the Shelby tube is remarkably lower than that by the Japanese sampler. The s_u value from the Japanese sampler is larger than that from the Shelby tube and nearly the same as that measured using the field vane test. However, the difference in the s_u values between the two samplers is less significant in the measurement of the laboratory vane, and for the direct shear test the difference in s_u cannot be identified. The reason for such difference in the s_u value for different shear tests may be attributed to the amount of the residual effective stress (suction) remained in the specimen. Tanaka (2000) has revealed that the losing residual effective stress does not necessarily cause the breaking of soil structure. Whether the soil structure is disturbed or not can be judged by the shape of the e - $\log p$ curve. It should be remembered that sample quality does not affect the e - $\log p$ curve as shown in Figs. 11 and 12. Therefore, this suggests that the structure is not greatly damaged by sampling even using the Shelby tube in the case of Hachirogata Clay. Although the suction of the specimen was not measured in this investigation, it is inferred that the residual effective stress for samples obtained by the Shelby tube is smaller than that by the Japanese sampler because of small unconfined compression

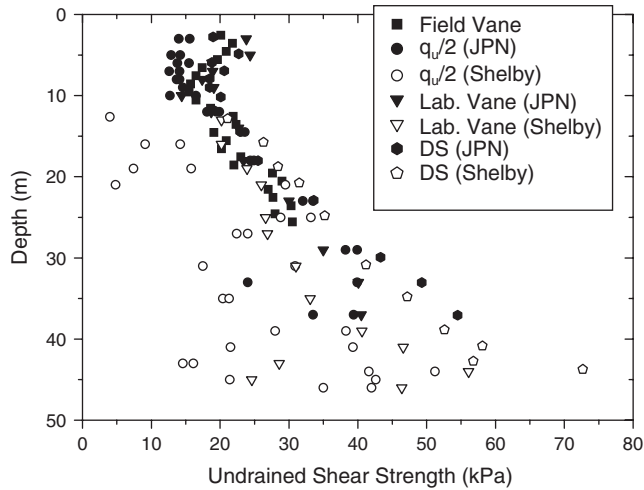


Figure 22. Comparison of undrained shear strengths measured by various shearing test for samples with different sample quality.

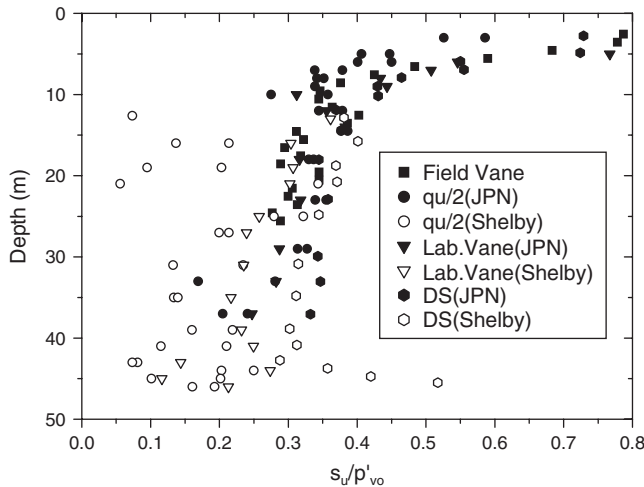


Figure 23. Normalized undrained shear strength by the in situ effective overburden pressure.

strength (q_u). However, it can be assumed that the soil structure is not significantly distracted since the same s_u values are measured for the Shelby tube, when the specimen is consolidated under p'_{vo} in the direct shear box. In the case of the laboratory vane, some part of the residual effective stress and confining stress remained even in the Shelby tube sample, because the specimen was confined by the sampling tube.

Figure 23 shows the undrained shear strength normalized by p'_{vo} . The influence of sample quality on the shear strength is much more prominent than that in Fig. 22. It can be seen in the figure that near the ground surface, the s_u/p'_{vo} is very large, although the p_c value is the same as p'_{vo} (see Fig. 13). The depths indicating large s_u/p'_{vo} also correspond to those where a lot of organic matter as well as diatoms are found. It is usually observed even in normal ground that the s_u/p'_{vo} ratio near the ground surface is relatively large. Such a large s_u/p'_{vo} is mainly attributed to high OCR created by fluctuation of the ground water table. However, as seen in Fig. 13, the OCR near the ground surface is not large enough to explain the high s_u/p'_{vo} . Therefore, it is inferred that the large s_u/p'_{vo} values may be partly attributed to the presence of organic matter or diatoms. However, below a depth of 15 m, the s_u/p'_{vo} ratio becomes constant and its value lies in the range of 0.3 to 0.34, excluding the strengths influenced by poor quality samples. As mentioned before, the OCR measure by CRS tests shows 1.2 for moderate depths. Consequently, the

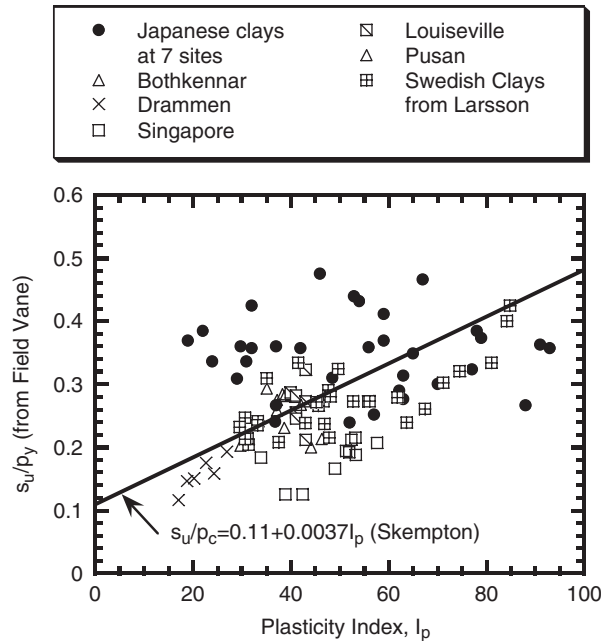


Figure 24. Relation between Strength incremental ratio and I_p . Data for Swedish clay is referred after Larsson (1991).

strength increment for consolidation pressure (s_u/p_c) is about 0.27. Tanaka (2000) has revealed that s_u/p_c measured by the field vane for Japanese clays is independent of I_p and relatively high compared with other clays, as shown in Fig. 24. Tanaka & Locat (1999) and Shiwakoti et al. (1999) have pointed out that one of the reasons for high s_u/p_c value for Japanese clays is the high content of microfossiles, especially diatoms.

6.2 Cone factor for CPT

Figure 25 correlates the net resistance ($q_t - p_{vo}$) from CPT with s_u from the unconfined compression and the field vane tests, where q_t and p_{vo} are, respectively, point resistance considering the effective area of the cone and the in situ total overburden pressure. The cone factor, N_{kt} ($s_u = (q_t - p_{vo})/N_{kt}$) ranges from 8 to 10, as indicated in Fig. 25. Although there is controversy as to whether N_{kt} is dependent on I_p , N_{kt} for Japanese clays seem to be independent of I_p , and range from 9 to 14, based on s_u from the field vane test (see Fig. 26).

6.3 Bjerrum's correction factor

Bjerrum's correction factor is widely used to modify s_u from the field vane test for application of the undrained shear strength to the total stress analysis for stability. On the other hand, Japan has traditionally used s_u evaluated from the unconfined compression test ($q_u/2$) without any correction factor. Figure 27 shows comparison between $q_u/2$ and μs_u , where μ is Bjerrum's correction factor determined by I_p (Bjerrum, 1973) and where s_u is measured by the field vane test. It is clearly observed in the figure that the ratio of $\mu s_{u(vane)}/(q_u/2)$ is dependent on I_p : i.e., at relatively small I_p , $\mu s_{u(vane)}$ is larger than $q_u/2$ (i.e., $\mu s_{u(vane)}/(q_u/2)$ ratio is greater than 1.0), while $q_u/2$ is larger when I_p exceeds 40. The reason for large $\mu s_{u(vane)}$ values may be due to underestimation of the s_u obtained by $q_u/2$. As already mentioned, Japanese clay consists mainly of clay minerals with high activity such as smectite. Therefore, soil with low I_p contains a lot of coarse materials such as silt or sand, consequently the suction in such soils cannot be retained when the soil is sampled and exposed to the atmosphere. The q_u value is strongly affected by the amount of the residual effective stress (suction) because the test is carried out under unconfined stress conditions. Therefore, it can be judged that the $q_u/2$ value for low I_p underestimates the undrained shear strength. It is interesting that for overseas clays (shown by open symbols in Fig. 27), which present

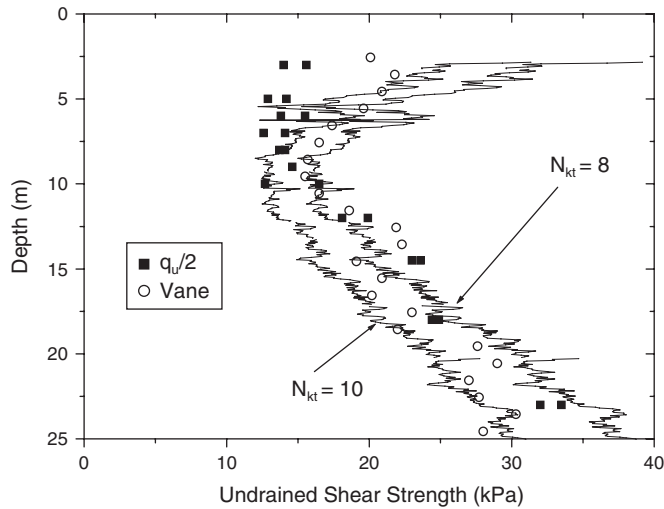


Figure 25. Cone factor for Hachirogata Clay.

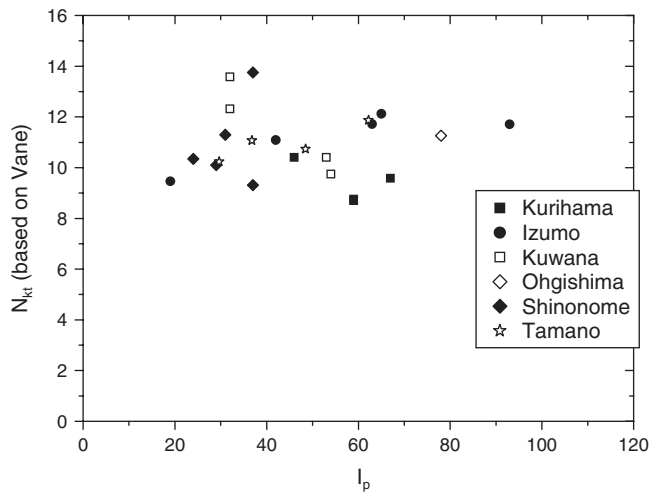


Figure 26. Cone factor for various Japanese clays.

relatively low I_p even with large contents of clay particles, their scatter in $\mu s_{u(\text{vane})}/(q_u/2)$ is small and their ratio is generally less than 1.0.

On the other hand, the $q_u/2$ is considerably larger than $\mu s_{u(\text{vane})}$ at moderate and large I_p (and for most overseas clays). This leads to the question “Does the unconfined compression test overestimate the true strength?” If the field vane strength modified by Bjerrum’s correction factor provides the true strength and the $q_u/2$ gives unsuitable strengths for stabilization analysis, many accidents of failures should have been reported because foundations are designed based on $q_u/2$ value in Japan, as mentioned before. However, such reports of such cases have never been published, neither has there been any reconsideration to change the design method using $q_u/2$. Rather, it should be concluded that the modification by the Bjerrum’s correction factor gives conservative s_u for foundation design, at least for Japanese soils. Taking a look at overseas cases, for example, as shown in Fig. 27, $\mu s_{u(\text{vane})}$ for Bothkennar, Pusan and Singapore clays is considerably smaller than their $q_u/2$ value even at small I_p . Although it is true that only Japan evaluates the undrained shear strength by the unconfined compression test, Bjerrum’s correction factor probably underestimates the undrained shear strength. The concept of the Bjerrum’s correction factor is derived for modifying the strain rate and the anisotropy effects: i.e., the rotation speed of the field

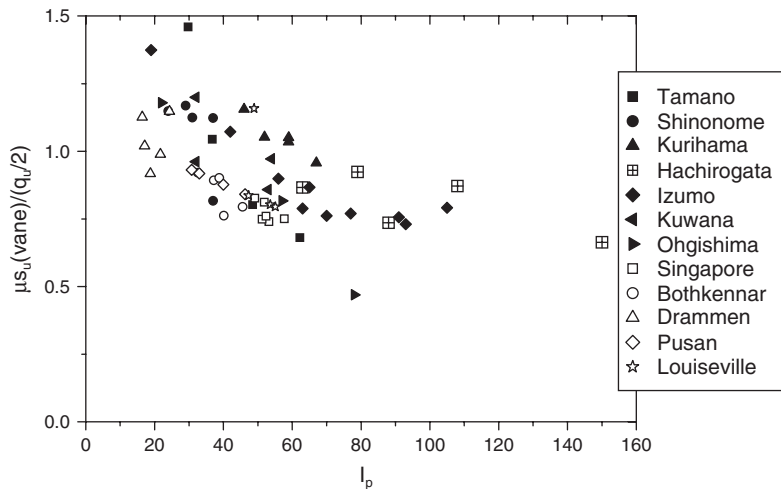


Figure 27. Comparison of undrained shear strength for design: Bjerrum's and q_u methods.

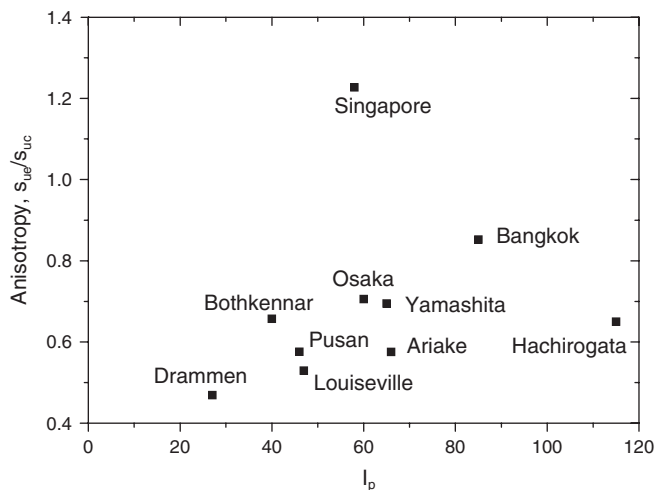


Figure 28. Anisotropy of undrained shear strengths. s_{ue} and s_{uc} are from extension and compression triaxial tests, respectively.

vane test is too fast compared with that in the field, and shear strength derived from the field vane test is mainly mobilized along the vertical plane. Bjerrum considered that these effects are simply correlated with I_p and he proposed a single line, which is derived based on I_p for modification of the vane strength. However, as already indicated in Fig. 21, the rate effect on the e - $\log p$ relation cannot be correlated with I_p : i.e., the rate effect does not increase with increase in I_p , unlike his assumption for deriving μ .

Figure 28 shows anisotropy in the undrained shear strength as the ratio of s_{ue}/s_{uc} , where s_{ue} and s_{uc} are the peak strength from extension and compression triaxial tests, respectively. The specimens in these tests were consolidated under pressures large enough to exceed their p_c value and following K_0 conditions. Although a tendency for s_{ue}/s_{uc} to increase with I_p can be slightly recognized, there is an explicit exception in these relations, i.e., Singapore clay. For this clay, the extension strength is larger than compression one.

It is recommended that before Bjerrum's correction factor is applied to sites without experiences, its validity should be examined carefully.

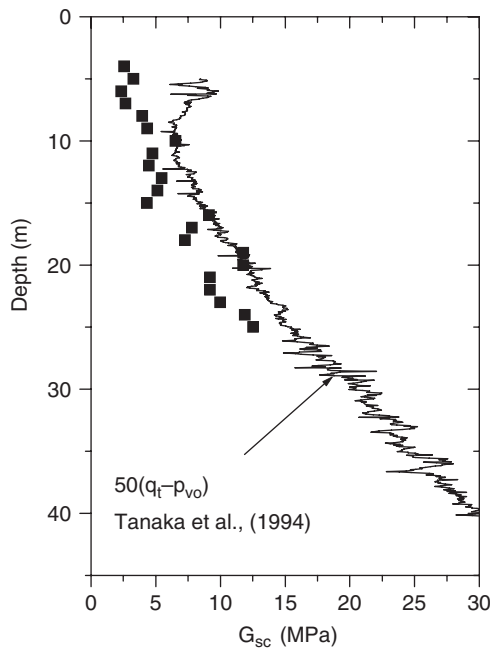


Figure 29. Shear modulus measured by seismic cone, compared with estimated value from CPT (Tanaka et al., 1994).

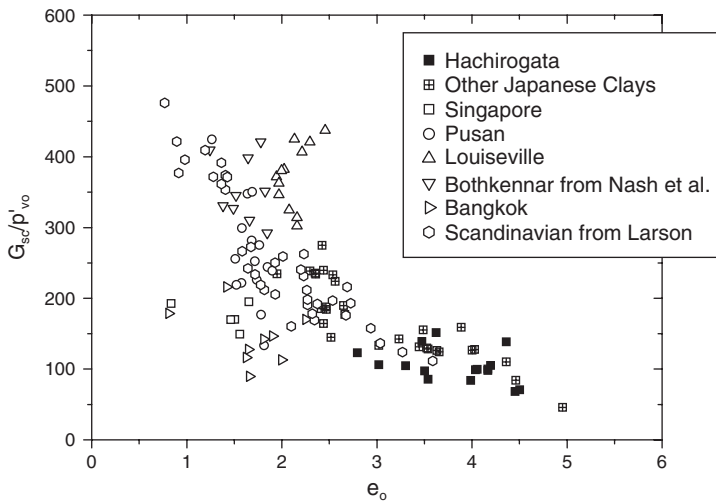


Figure 30. Ratio of shear modulus to the effective overburden pressure versus void ratio.

7 SHEAR MODULUS MEASURED BY SEISMIC CONE

Shear modulus for Hachirogata was measured by a seismic cone (G_{sc}). The result is shown in Fig. 29, together with estimated G_{sc} from the CPT using an empirical relation $G_{sc} = 50(q_t - p_v)$ (Tanaka et al., 1994). From the figure, the estimated G_{sc} is somewhat larger than the measured one.

Until now, many researchers have investigated what factors govern the G_{sc} value. Figure 30 shows the relation between G_{sc}/p'_{vo} and in situ void ratio (e_o). Although the variation in G_{sc}/p'_{vo} becomes large

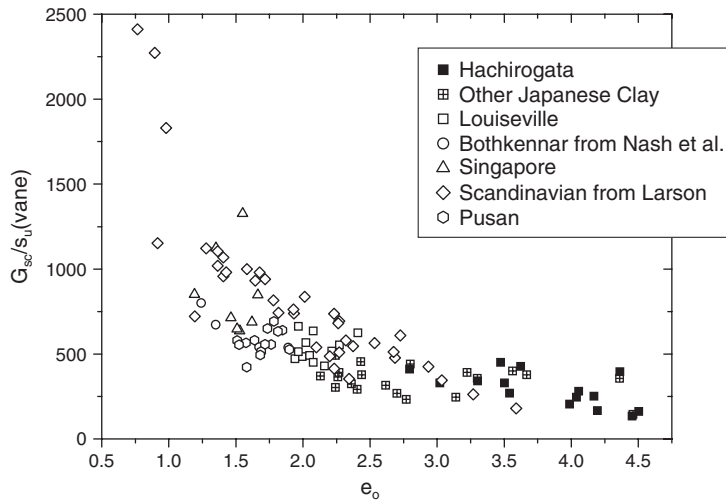


Figure 31. Ratio of shear modulus to the vane shear strength versus void ratio.

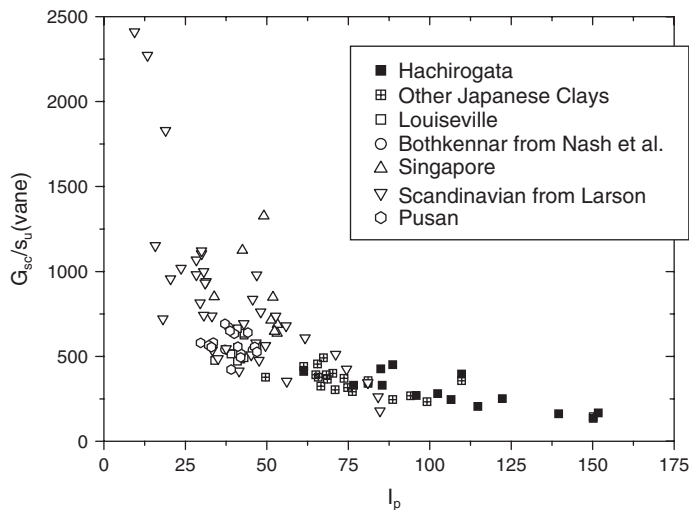


Figure 32. Ratio of shear modulus to the vane shear strength versus plasticity index.

at small e_o , especially less than 2.5, there exists an explicit tendency: with an increase in e_o , G_{sc}/p'_{vo} becomes smaller. Especially for Japanese clays including Hachirogata Clay, the relation between G_{sc}/p'_{vo} and e_o is clearly observed. The reason for the large scatter of G_{sc}/p'_{vo} for overseas clays is not clearly identified; however, one of the key factors for such large scatter is variation in OCR. It is reasonable to consider that as OCR increases, G_{sc}/p'_{vo} should increase if other conditions are identical. In this sense, the normalization of G_{sc} by s_u measured from the Field vane test, for example, seems much more reasonable than that by p'_{vo} , because s_u is also influenced by OCR. The relation between $G_{sc}/s_{u(vane)}$ is shown in Fig. 31, plotted against e_o . Compared to that in Fig. 30, the scatter at lower e_o in $G_{sc}/s_{u(vane)}$ is much smaller than in G_{sc}/p'_{vo} , and the relation for overseas as well as Japanese clays converges within a relatively narrow band. When e_o is larger than 2.5, the $G_{sc}/s_{u(vane)}$ slightly decreases with e_o . On the other hand, at e_o smaller than 2.5, $G_{sc}/s_{u(vane)}$ considerably increases with decrease in e_o .

Figure 32 plots relation of the $G_{sc}/s_{u(vane)}$ against I_p , instead of e_o , because e_o is not an inherent value, but is influenced by consolidation pressure, i.e., depth or OCR. However, the appearances of these two figures are not much different regardless of whether e_o or I_p is plotted. Since the depths at the sites used

for these figures are somewhat restricted (the maximum depth is at most 45 m), e_o and I_p are strongly related. That is, if I_p is large, e_o is also inevitably large. Nevertheless, the degree of scatter in $G_{sc}/s_{u(vane)}$ against I_p is much smaller at large I_p . However, at small I_p , the data scatters in both Figs. 31 and 32 are almost identical.

8 CONCLUSIONS

From a series of site investigations at the Hachirogata site, where thick clay with extremely large plasticity Index (I_p) is deposited, the following interesting facts are found.

- 1) It is inferred that the reasons for large I_p of Hachirogata Clay are: clay mineral with large activity such as smectite; high content of organic matter and a large amount of macrofossils, especially diatoms.
- 2) Two different sample qualities were prepared, i.e., using a Japanese standard sampler with a stationary piston and a Shelby tube without a piston. The e - $\log p$ curves for soils with two different sample qualities are nearly identical. However, there is a large difference in the unconfined compression strength (q_u) between the Japanese sampler and Shelby tube. This difference may be caused by difference in the residual effective stress (suction) in samples collected by different samplers.
- 3) The yield consolidation pressure (p_c) is slightly larger than the in situ effective overburden pressure (p'_{vo}). At moderate depths, the overconsolidation (OCR) ratio is about 1.2.
- 4) The compression index at large consolidation pressure (C_{c1}) can be well correlated with liquid limit (w_L): $C_{c1} = 0.009(w_L - 10)$.
- 5) Undrained shear strength normalized by p'_{vo} (s_u/p'_{vo}) near the ground surface is extremely large even though the soil is not so overconsolidated. This large s_u/p'_{vo} is inferred to be partly attributed to a large amount of organic matter and diatoms. At greater depth where the influence of organic matter and diatoms is insignificant, the s_u/p'_{vo} reduces to a more usual value, i.e., around 0.32.
- 6) The cone factor for piezocone (N_{kt}) is between 8 and 10, this range falls within the value of 9 to 13 for Japanese marine clays.
- 7) Bjerrum's correction factor provides conservative undrained shear strength, compared with the unconfined compression strength, which is extensively used in Japan. It is recommended that before Bjerrum's correction factor is applied to sites without experience, its validity should be examined carefully.
- 8) The shear modulus obtained by the seismic cone (G_{sc}) is normalized by p'_{vo} or s_u from the vane test, (G_{sc}/p'_{vo}) or ($G_{sc}/s_{u(vane)}$). Both G_{sc}/p'_{vo} and $G_{sc}/s_{u(vane)}$ are dependent on the in situ void ratio (e_o).

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In-situ test calibrations for evaluating soil parameters

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ABSTRACT: The interpretation of in-situ geotechnical test data needs a unified approach so that

soil parameters are evaluated in a consistent and complementary manner with laboratory results. A

common thread in assessing in-situ tests is the focus on the geologic stress history, often expressed

by the overconsolidation ratio (OCR). For clays, the OCR can be measured by consolidation tests

on undisturbed samples, yet for sands is rather problematic to address. For 6 clays, a hybrid cavity

expansion - critical state model is used to match responses measured CPT, CPTu, and DMT. Specifically,

tip stress, sleeve friction, penetration porewater pressures, and flat dilatometer readings are fitted by

parametric input of OCR, void ratio, friction angle, rigidity index, and compressibility parameters. The

undrained shear strength (s_u) of clays is best handled via critical-state concepts. Discussions are included

for pressuremeter, vane, and T-bar tests. For sands, select empirical methods derived from laboratory

chamber testing on reconstituted clean quartz and siliceous sands are reviewed, specifically for effective

friction angle ϕ' , OCR, and K_0 . In a novel look, a special set of undisturbed (frozen) sand samples

from 15 locations in Japan, Canada, Italy, Norway, and China is used to check interrelationships for the

following in-situ penetration tests: SPT, CPT, and V_s . Stiffness of all soils begins with the small-strain

shear modulus ($G_0 = G_{max} = \rho T V_s^2$) that can be used together with strength (s_u or ϕ') to evaluate stiffness

over a range of strains. Supplementary testing by PMT and/or DMT can provide intermediate stiffnesses

for tuning of modulus reduction schemes, as well as independent assessments of K_0 and OCR.

1 INTRODUCTION

1.1 Natural geomaterials

Geomaterials encompass an extremely wide range of natural soils, rocks, and intermediate earthy sub

stances, as well as artificial fills, mine tailings, and slurries (see Figure 1). Grain sizes of these materials

vary tremendously from nano-particles in the very small angstroms domain and colloidal range (<0.5

microns), to clay particle sizes (<2 microns), silts (<75 microns) to coarse-grained soils (>0.075 mm)

including sands to gravels, as well as much larger cobbles (150 to 300 mm) and boulders (>300 mm)

to fractured rocks and massive intact rock formations (1 m to 1 km). These geomaterials are generally

very old (thousands to millions of years old) with the youngest generally represented by Holocene age

formations that have been placed only within the last several thousand years (<10,000 years).

Natural geomaterials have been formed by an assortment of geologic processes including water sedi

ment (marine, lacustrine, estuarine, alluvial, deltaic, fluvial), ice (glacial), wind (aeolian), disintegration

(residual), mass wasting (colluvial), chemical (carbonates, calcarenites), biological (organic), and other

mechanisms (e.g., meteoric). The varied mineralogies (e.g., quartz, feldspar, mica, chlorite, illite,

smectite, kaolinite, montmorillonite, hallosite, carbonates) can occur in unlimited combinations and

permutations, thus forming infinite possible grain size distributions, as well as variances in plasticity, par

ticle shape, size, roundness, packing, porosity, and fabric. These enumerable facets could be construed

to defy any notion that characterization of these materials is at all possible.

After initial placement or forming, these materials are subjected to a wide range of environmental,

geomorphological, and climatic changes including erosion, desiccation, ageing, wet-dry cycles, seasonal

temperature fluctuations, and in some cases, special events such as glaciation, cyclic loading due to

earthquakes, load fluctuations due to wave and tidal activities, and other activities. As a consequence,

most soils are at least slightly to moderately overconsolidated to some degree. The significant fact that

Figure 1. Illustrative examples of diversity in natural geomaterials found on planet Earth.

sea level has risen 30+ m in recent geologic times (thus associated groundwater tables), is sufficient

alone in causing appreciable preconsolidation in soils (Locat, et al. 2003).

1.2 Site investigations

A proper characterization of natural geomaterials is paramount in all site investigations because the

results will impact the solution with respect to safety, performance, and economy. For instance, if the

site characterization program has collected poor quality undisturbed samples of clays & silts for lab

oratory consolidation and triaxial shear testing, the soil stiffness and strength will be underestimated,

perhaps resulting in the requirement of driven pile foundations for the building structure whereas shallow

footings could have adequately sufficed. In site investigations of clean sands that are most difficult to

sample, the assumption of normally-consolidated conditions will inevitably lead to an underestimate in

pile side friction or an overestimate in shallow footing settlements. Therefore, a knowledge of in-situ

test methods and their interpretation is of great value to the practicing geotechnical engineer in order to

place the best value in the applied solution in site development. A number of different approaches are available for the assessment of ground conditions at a particular

site. These might be grouped into three categories, in preferred order:

A. Geophysics for general mapping of the relative variances across the site.

B. In-situ tests for profiling vertical geostratigraphic

changes and/or soil parameter evaluation.

C. Drilling and sampling to obtain high quality and representative materials for the laboratory. An integrated approach should be adopted for studying the ground whereby non-destructive geo

physics are used first to guide the selection of expedient probe sounding locations, which in turn would

aid in the choice & direction of the more expensive & laborious soil test borings to provide undisturbed

samples for controlled laboratory testing by triaxial, simple shear, consolidation, and/or resonant col

umn devices. A versatile and well-calibrated constitutive soil model would link these aspects together

in a rational and consistent framework. A full discussion on the basic laboratory, field, & geophysical

methods and their procedures is beyond the scope of this paper but may be found elsewhere (e.g., Mayne,

Christopher, & DeJong, 2002). A large number of in-situ devices and field tests are available to delineate the geostatigraphy and

determine specific engineering parameters in the ground. As depicted by selected tests in Figure 2, these

SPT

TxPT LPT VST PMT CPMT DMT SPLT K θ SB Swedish Weight
Sounding HF BST TSC FTS CPT CPTu RCPTu SCPTu SDMT T-bar
Ball Plate CHT DHT SASW MASW CSW Suspension Logging SPTT

Figure 2. Selection of available in-situ geotechnical tests for determination of soil parameters.

are quite numerous and include: standard penetration, cone penetration, dilatometers, pressuremeters,

vanes, flat and stepped blades, hydro-fracture, borehole shear, torsional probes, and many other innova

tive designs. Robertson (1986) provides a detailed overview on many of the standardized and specialized

devices that are primarily penetration type and/or direct-insertion type devices for testing the ground.

Woods (1978) and Campanella (1994) give a review of applicable geophysical tests for determining

mechanical wave properties. Wroth (1984, 1988), Jamiolkowski et al. (1985), and Lunne et al. (1994)

provide summary papers concerning the use & interpretation of the more common standardized tests

and recent updates have been made by Yu (2004), Mayne (2005), and Schnaid (2005). An optimization

of site-specific site investigation is achieved with seismic piezocone testing (Campanella et al. 1986)

together with intermittent dissipation testing (SCPT_i), since five independent readings are obtained with

depth in the same sounding (Mayne & Campanella 2005): q_t , f_s , u_b , t_{50} , and V_s . A similar procedure can

be provided by the seismic dilatometer test (Mlynarek et al. 2006) with A-reading dissipation (SDMT_a)

to obtain: p_0 , p_1 , p_2 , t_{flex} , and V_s .

1.3 Conventional approach for clays

In the conventional and classical methods of interpretation, the results of in-situ tests are commonly

divided into two categories: (a) clays, whereby the undrained shear strength (s_u) is assessed; and (b) sands,

whereby the relative density (D_R) and/or effective friction angle (ϕ') is evaluated. Yet, within the frame

work of critical state soil mechanics (CSSM), all soils in fact are frictional materials and their strength

envelope can be best represented by their effective stress friction angle (ϕ'). For most soils, it can be

taken that the effective cohesion intercept $c' \approx 0$ (unless true cementing or bonding is present). As these

geologic materials are quite old and have been lying

uninterrupted for long eons of time, the drained

strength, in most cases, is quite characteristic of their "normal" behavior. With the introduction of man's

activities involving construction on the ground, however, short term conditions can result in what might

be termed "undrained" loading, corresponding to shearing under constant volume. This undrained con

dition is critical for normallyto lightly-overconsolidated soils of low permeability (i.e., soft clays &

silts) under relatively fast (geologically speaking) rates of loading. Notably, the undrained conditions

are merely transient, and given time, excess porewater pressures will eventually dissipate to equilibrium

conditions (i.e., hydrostatic porewater pressures, $u = 0$).

For clays, one difficulty with the conventional interpretation methods involving in-situ tests is that

a reference benchmark value of s_u is required in the calibration and/or verification of direct probe

tests, such as the standard penetration test (SPT), cone penetration test (CPT), piezocone (CPTu), flat

dilatometer test (DMT), and other devices (e.g., T-bar, ball penetrometer). The dilemma is depicted in

Figure 3. The appropriate mode of undrained shearing is not always known at the time of interpretation

and might include triaxial compression (CAUC, CK $\emptyset$ UE, CIUC), plane strain compression (PSC), simple

shear (SS), torsional shear (TS), or one of the extension modes (CAUE, CIUE, PSE, CK $\emptyset$ UE). Moreover, Undrained shear strength of clays $\sigma_c \approx c_u \approx s_u \approx \tau_{max}$ Classical interpretation of CPT and VST in clays: $N_{kt} \approx q_t \approx \sigma_{vo} \approx s_u$ Which s_u ? HC CIUC CK $\emptyset$ UC DSC DSS DS PSE CK $\emptyset$ UE CIUE UU UC μ vane $\approx$ fctn(PI) $\approx 7 \text{ md } 3 \text{ 6T } S_u \text{ CORR } \approx \mu_v \cdot N_{KCPT} \approx 15 \pm 7 \text{ PSC}$

Figure 3. Dilemma in matching laboratory benchmark mode for undrained shear strength (s_u) with in-situ CPT

and VST data. Note: Direct simple shear (DSS) is the likely reference for stability problems.

multi-modes or non-standard modes may actually apply. In consideration of these possibilities, it is not

at all clear how stress state, anisotropy, and direction of loading affects the interpretation of s_u from each

of the field tests. Another problem lies in the inconsistency amongst interpretative frameworks for in-situ tests. Specif

ically for clays, the vane shear test (VST) is analyzed using limit equilibrium methods, whilst the

pressuremeter test (PMT) is assessed within cylindrical cavity expansion theory, and yet the bearing fac

tor term for interpretation of the cone penetration test (CPT) might come from either strain path method

or finite elements solutions. For a given parameter (i.e. s_u), the different theories alone will provide

inconsistencies in interpretations amongst the various in-situ tests, within the same soil formation. Each of the in-situ test types is conducted at a different rate of strain, thus affecting compatibility in the

comparison of different tests. For example, at a rate of penetration of $v = 20 \text{ mm/s}$, the CPT pseudo-strain

rate could be considered to be v/d , where the diameter of a standard 10 cm ϕ penetrometer is $d = 35.7 \text{ mm}$.

This pseudo-strain rate is 56%/s, or about $2 \cdot 10^5 \text{ \%/hour}$. In comparison with a standard laboratory triaxial

compression test performed at 1%/hour, this is over 5 orders of magnitude faster! For the aforementioned

reasons of strength anisotropy, strain rate, boundary conditions, as well as other factors (e.g., progressive

failure, drainage, fissuring), empirical factors have been included in the interpretation of undrained

strengths from in-situ tests. Notably, the vane shear

shows the various derived undrained shear strengths from

laboratory triaxial compression, simple shear,

and extension tests on undisturbed samples (Hight et al. 2003) together in comparison with direct

measured values from self-boring pressuremeter (Powell & Shields, 1995) and field vane tests (Nash,

et al. 1992). Quite a variation is seen for all depths. Also indicated is the backfigured operational value

of s_u from a full-scale footing load test ($B = 2.4$ m) conducted at the site using a limit plasticity solution

with a bearing factor $N * c = 7$, as reported by Jardine, et al. (1995).

1.4 Conventional approaches for sands

For sands, the major difficulty lies in the inability to collect true undisturbed samples from the field in

order to establish a clear laboratory calibration value of strength or stiffness whatsoever. Therefore, pri

mary efforts have resorted to collecting disturbed bulk samples of sands from the field and reconstituting

the sand specimens at the "same density" in the laboratory. Presumably, the in-place density has been

accurately assessed and the lab results give stress-strain-strength behavior that can be linked to the field

tests. The reconstituted sands can either be formed into:
(a) small triaxial or direct shear-size specimens

that are related back to field penetrometer readings, or
(b) large diameter calibration chambers, mostly

of the flexible-wall type, that allow the full probing by the penetrometers or in-situ tests. Dry to saturated

sands under normally-consolidated to induced overconsolidated states have been prepared in this manner

under different boundary conditions (Ghionna & Jamiolkowski 1991). After all is known beforehand

about the chamber deposit of sand (particle shape & angularity, median particle size, percent fines, density, stress history), a miniature to full-size probe is inserted for the test performed (Figure 5). Lab

oratory calibration chamber tests (CCT) have included SPT, CPT, DMT, PMT, CPMT, and mechanical

wave geophysics for compression or P-wave and shear or S-wave velocity measurements.

It is well-known that the CCT data suffer from their limited boundary sizes represented by the ratio D/d ,

where D = diameter of large sand sample and d = diameter of probe. Thus, the results need be corrected

to far-field values (e.g., Salgado, et al. 1997; Jamiolkowski et al. 2001). Moreover, different preparation

methods for placement of “sand deposits” have been used in CCT, including: compaction, air pluviation,

water pluviation, moist tamping, vibration, and slurrying. For a given density of the sand, these in fact

do not provide similar fabrics and thus the obtained probe results may differ significantly from field

behavior (e.g., Hoeg et al. 2000; Jamiolkowski 2001). Finally, the supposed link in the relationships

derived from the CCT correlations derives from the assumption that the in-situ tests can provide a direct

measure of the relative density (D_R). In US practice, the notion of D_R only applies to cases of clean

Figure 5. Calibration chamber testing setup for in-situ testing on reconstituted sands.

sands with percent fines $PF \leq 15\%$. In truth, the use of in-situ penetration measurements can provide

only a rough index on the in-place packing arrangement such as void ratio, porosity, or relative density.

Thus, other devices such as gamma logging, downhole nuclear gages (i.e., radio-isotope penetrometers),

or time domain reflectometry (TDR) in direct push probes would serve better for assessing this initial

state condition.

1.5 Fitting & calibration approaches

Herein, a couple of non-conventional approaches are presented for evaluating in-situ test results in clays

and sands. The emphasis is initially on the CPT, CPTu, and DMT for clays, with dissipation response

also considered; and SPT, CPT, and V s testing in sands. The methods are not rigorous but utilized to

show an important geotechnical need regarding a full set of complementary calibrations of in-situ tests

within a soil medium. For the clays, a known profile of stress history (i.e., OCR) is used to drive the fitting of in-situ

penetration data in terms of simple closed-form expressions derived from hybrid cavity expansion -

critical state models. Input parameters include the initial state (void ratio, e_0), effective friction angle

(ϕ'), plastic volumetric strain potential ($= 1 - C_s / C_c$), and rigidity index ($I_R = G/s_u$). Reasonably

successful forward profiles of cone tip stress (q_t), penetration porewater pressures (u_1 and/or u_2), sleeve

friction (f_s), dilatometer contact (p_0) and expansion pressures (p_1) are shown for three soft clay sites and

two overconsolidated clays. Research needs include a generalized methodology for all types of in-situ

tests that is internally consistent and then calibrated with well-researched test sites, as those reported in

the 2003 and 2006 Singapore Workshops (Tan, et al. 2003). For sands, a two phase study is given: (1) examination of selected existing relationships for stress

history (K_0 and OCR) and friction angle (ϕ') at three

documented sand sites; and (2) a specially

compiled triaxial database created from expensive undisturbed (primarily frozen) sand samples where

in-situ standard penetration, piezocone penetration, and shear wave measurements were collected. These

latter series document the measured void ratio (e_0), relative density (D_R), triaxial friction angle (ϕ'), and

stiffness from stress-strain curves. The parameters can then be compared with their corresponding in-situ

measurements of N-value, tip stress q_t , and downhole V_s values in terms of existing correlative trends,

theoretical relationships, and/or to allow future calibration with soil constitutive models.

2 IN-SITU TESTS IN CLAYS

2.1 Overview

In this approach, simple analytical models will be matched to the measured in-situ field data based

on its stress history profile. Soil input parameters are parametrically varied to provide the optimal

fitting to all available tests. At this time, the author has experimented with a simple hybrid model based

primarily on spherical cavity expansion and critical-state soil mechanics (CSSM) that can be applied

to cone penetration, porewater pressures, and flat dilatometer tests in clays. Brief discussions are given

subsequently on the topics of stress history, cavity expansion, and CSSM.

2.2 Stress history

The benchmark for the preconsolidation stress is that caused by mechanical processes, specifically the

removal of overburden, as occurs by erosion, glaciation, and/or excavation (Chen & Mayne 1994; Locat

et al. 2003). Yet, effects of overconsolidation that are caused by a rise in the groundwater table, desicca

tion, wet-dry cycles, freeze-thaw cycles, and/or quasi-preconsolidation due to secondary consolidation,

creep, and/or ageing might also be ascertained from these approaches, at least on an approximate basis.

For clays, the preconsolidation stress ($\sigma'_p = \sigma'_{vmax} = P'_c$) can be uniquely determined as the yield point

on conventional one-dimensional consolidation plots of void ratio vs. log effective stress (i.e., e -log σ'_v

data). The normalized form is termed the overconsolidation ratio, $OCR = (\sigma'_p / \sigma'_{vo})$. Because of sample

disturbance issues, the clear demarcation of σ'_p may be unclear or muddled, thus graphical "correction"

schemes have been devised to better delineate its value (e.g., Grozic and Lunne 2004).

The OCR governs both the normalized undrained shear strength (s_u / σ'_{vo}) and the lateral geostatic

stress coefficient, $K_0 = (\sigma'_{ho} / \sigma'_{vo})$. The OCR has also been shown especially influential on laboratory

derived values of small-strain shear modulus ($G_0 = G_{max}$), Skempton's porewater parameter (A_f), and

intermediate stiffness values of modulus and rigidity (E_u / s_u and E' / p'_{00}). For natural sands, the effective

preconsolidation occurs similarly by the same basic mechanisms. In calibration chamber tests involving

sands, the prestressing is induced artificially as part of creating the "deposit", thus the OCR is known.

In natural sands, however, it has been difficult to ascertain the OCR because of sampling problems and

the very flat e -log σ'_v response of sands in one-dimensional compression tests.

In truth, the vertical preconsolidation stress is just one

point of an infinite locus of memory on the three

dimensional yield surface (Leroueil & Hight, 2003). As such, it is possible to define additional points of

yielding along specialized stress paths in the triaxial apparatus and define this complete yield surface.

This yield surface is rotated and centered about the K_0 NC compression line in MIT type q - p space and

governed by the effective frictional characteristics of the soil (e.g., Diaz-Rodriguez, Leroueil, & Aleman,

1992). Thus, in sands, it is conceptually feasible to use the same penetration data (N_{60} , q_t , p_0) to define

the centering of the yield surface, as well as the friction angle of the material, since the two are related.

For clays, the in-situ data (VST, CPT, CPTu, SPT, DMT) relate to the effective preconsolidation stress,

thus also to the yield surface. However, since these are relatively fast tests under undrained conditions

($\Delta V/V_0 = 0$), then additional data related to the generated excess porewater pressures are needed to

define the effective frictional envelope (e.g., Senneset et al. 1989).

2.3 Cavity expansion theory

In cavity expansion, three possible geometric cases can arise during integration of the governing equa

tions: (a) linearly, to create a wall; (b) radially, to form a cylinder; and (c) spherically, resulting in a ball.

The use of cylindrical cavity expansion (CCE) theory for the reduction of pressuremeter test (PMT) data

in clays has been well-appreciated for over 50 years (Gambin, et al. 2005). Assuming undrained condi

tions, four parameters can be individually assessed for each PMT including: lift-off pressure ($P_0 = \sigma_{ho}$),

elastic shear modulus (G), undrained shear strength (s_u),

and the limit pressure (P_L), such that:

where $I_R = G/s_u$ = undrained rigidity index. Thus, all four parameters are interrelated via equation (1).

Figure 6 shows a comparison of the graphically-interpreted limit pressure versus that calculated using (1)

with data from 34 clays tested by self-boring pressuremeters (SBP), thus indicating the general validity of CCE.

Details on the PMT interpretative procedures are given elsewhere (e.g., Wroth, 1984; Briaud, 1995;

Ballivy 1995; Fahey & Carter, 1993; Clark & Gambin, 1998). Alternate approaches for cavity expansion 100 1000 10000 CCE: $P_L = P_0 + s_u (1/I_R)$ (kPa) Measured PMT P_L (kPa) 5 Fissured clays 29 Intact clays SBPMT Database from Mayne & Kulhawy (1990)

Figure 6. Measured vs. predicted limit pressure using SBPMT database in clays.

interpretation include the derivation of a continuous stress-strain curve from the measured pressure vs.

volume change field data (e.g., Gambin et al. 2005). The cavity expansion approach has been also extended to drained loading conditions by consideration

of volumetric strain changes (Vesic, 1972, 1977), or alternatively using dilatancy angle (Carter, et al.

1986). Additional work has been undertaken to evaluate the unloading cycles during full-displacement

type pressuremeters, such as the cone pressuremeter test (CPMT), as detailed by Houlsby & Withers

(1988) and Yu (2004). A parameter of interest to all cavity expansion problems is the rigidity index, defined as the ratio

of shear modulus to shear strength, $I_R = G/\tau_{max}$. The appropriate value of I_R is rather elusive since the

shear modulus changes dramatically from an initial value ($G_0 = G_{max}$) to a secant value at peak strength,

$G_{min} = \tau_{max} / \gamma_f$, where γ_f = strain at peak. The tangent G is zero at peak strength. Notably, in SBPMT, the

value of G could be reported from the first-time loading (i.e., backbone curve), or else from an applied

unload-reload cycle (G_{ur}) that is considerably stiffer than the former. For penetration tests, the value is

likely closer to G_{min} . As illustrated by Figure 7, the rigidity index can also be considered as the reciprocal

of the failure strain, or $I_R = 1/\gamma_f$.

2.4 Critical-state soil mechanics

Critical state soil mechanics (CSSM) is a valuable effective stress framework to interrelate concepts of

frictional strength and consolidation (e.g., Schofield & Wroth, 1968). By specifying the initial state,

CSSM can easily explain the differences between normally-consolidated (NC) and overconsolidated

(OC) behavior, contractive vs. dilative response, undrained vs. drained strengths, positive vs. negative

porewater pressure generation, as well as other behavioral facets such as partly drained to semi-undrained

cases, cyclic behavior, creep, and strain rate (Leroueil & Hight 2003). In the most simple version involving saturated soils, only three soil properties are considered (ϕ' , C_c ,

C_s) in addition to the initial state (e_0 , σ'_{vo} , and $OCR = \sigma'_P / \sigma'_{vo}$). Essentially, CSSM is the link between

the well-known compression curves from one-dimensional consolidation tests (termed e - $\log \sigma'_v$ space)

and the Mohr's circles from triaxial or direct shear tests (represented by shear stress vs effective normal

stress plots, or τ - σ'_v space). These two spaces are depicted in Figure 8, along with a third diagram (e - σ'_v

space) that really offers no new information, just a way to follow the compression results in arithmetic

scale for $\sigma' v$ and tie to the $\tau - \sigma' v$ space. In its essence, the premise for CSSM is that all soil, regardless of starting point or drainage conditions,

strives towards and eventually ends up on the critical state line (CSL). In the $\tau - \sigma' v$ space, this line

corresponds to the well-known strength envelope given by the Mohr-Coulomb criterion for $c' = 0$. Here, τ shear stress γ shear strain τ_{max} s_u c_u undrained shear strength τ/γ G shear modulus Define: Rigidity Index: $I_R = G/s_u \gamma_{Ref} = 1/I_R G_0$

Figure 7. Shear stress vs. shear strain for soils and definitions of τ_{max} , G , γ_f , and I_R . Simplified Critical State Soil Mechanics Log $\sigma' v$ Effective stress $\sigma' v$ Effective stress $\sigma' v$ Shear stress τ

V_o

$i d$

R

$a t$

$i o$

, e Void Ratio, e_N NC $C C C S L C S L C S L C S$
 $\sigma_p' \sigma_p' OC$

Four Basic Stress Paths:

1. Drained NC (decrease V/V_o)
2. Undrained NC (positive u)
3. Undrained OC (negative u)
4. Drained OC (increase V/V_o) ϕ' e NC consolidation welling e OC 12 3 4 Yield Surface
dilatative contractive Δu Δu $\sigma_{CS}' \approx \sigma_p' \tau_{max}$ s_u NC
 τ_{max} $\sigma' \tan \phi' s_u OC$ τ_{max} $c' \sigma' \tan \phi' c' \Delta \Delta \Delta$

Figure 8. Simplified critical state soil mechanics, showing four common stress paths.

shear strength is represented by the maximum shear stress ($\tau_{\max}$) and is given by: $\tau = c' + \sigma' \tan \phi'$. In the

e - $\log \sigma' - v$ space, the CSL represents a line parallel to the virgin compression line (VCL with slope C_c) for

NC soils, yet offset to the left at stresses approximately half those of the VCL. All NC soils start on the

VCL, while all OC soils begin from a preconsolidated condition along the recompression or swelling line

(given by slope C_s). Regardless, all shearing results in peak stresses falling on the CSL. Figure 8 shows

a summary of four basic stress paths corresponding to drained loading (no excess porewater pressures,

or $\Delta u = 0$) and undrained loading (constant volume, or $\Delta V/V = 0$) for both NC and OC initial states. Surprisingly, geotechnical practice has still not adopted the framework of critical-state soil mechanics

(CSSM), yet the supporting research and clear evidence have been available for over a half-century (e.g.,

Hvorslev, 1960). A review of geotechnical textbooks used in the USA, for example, reveals that the most

popular and best-selling books do not discuss nor even mention CSSM, the exceptions being those by

Budhu (2000) and Lancellotta (1995). Of course, there are several excellent books specific to CSSM

(Schofield & Wroth, 1968; Wood, 1990), yet omitted in geotechnical education in USA, China, and

many other countries. With a lack of background in CSSM, many practitioners still cling to the incorrect notion of running

strength tests on three specimens to produce a set of total stress “ c and ϕ ” parameters. For clays, the idea of

“ $\phi = 0$ ” analysis still prevails in practice, yet all soils (clays, silts, sands, gravels) are frictional materials.

The observation of “ $\phi = 0$ ” is really an illusion when porewater pressures are not monitored or not known,

thus missing the CSL influence. As a consequence, the term “cohesion” is often used ambiguously,

in some instances referring to the undrained shear strength ($c = c_u = s_u$), yet in other circumstances,

intending to mean the effective cohesion intercept (c'). The former is obtained as the peak shear stress

in a stress path of constant volume. The latter is obtained by force-fitting a straight line ($y = mx + b$) to

represent the Mohr-Coulomb strength criterion ($\tau = \sigma' \tan \phi' + c'$) from laboratory strength data. As noted previously, the strength envelope is actually more complex and best described by a frictional

envelope (ϕ') having a superimposed three-dimensional curved yield surface that is governed by the

preconsolidation stress, σ'_P (Leroueil & Hight, 2003). The shape, size, features, and movement of the

yield surface distinguishes one constitutive model from another (e.g., Lade 2005), yet this facet is not

necessary in order to convey the overall simplicity and elegance of CSSM (e.g., Lancellotta, 1995). In

Figure 8, the basic concept of a yield surface is shown in the $\tau - \sigma' - v$ space. It is the intersection of this yield

surface with the frictional envelope which dictates the offset distance of the CSL to the left of the VCL.

2.5 Undrained shear strength

The undrained shear strength is best represented in normalized form (s_u / σ'_{vo}) which is mode-dependent

and reliant on initial stress state, strain rate, direction of loading, degree of fissuring, and other factors

(Kulhawy & Mayne, 1990). The ratio s_u / σ'_{vo} can be evaluated using constitutive laws based on critical

state soil mechanics (e.g., Wroth, 1984; Ohta et al. 1985; Whittle & Kavvas, 1994), or alternatively

using empirical approaches such as SHANSEP (Ladd, 1991). For general use, the author subscribes

to a three-tiered hierarchy, based on the availability of site-specific data (Mayne, 2003). Since direct

simple shear (DSS) provides an overall representative mode for stability and bearing capacity analyses,

the preferred approach is via CSSM (Wroth, 1984), as illustrated by Figure 9 and expressed by:

where $\phi' =$ effective friction angle, $= 1 - C_s / C_c$ is the plastic volumetric strain ratio, and C_s and C_c

are the swelling and virgin compression indices, respectively. For intact natural clays, the full range

of observed frictional characteristics vary from $18^\circ < \phi' < 43^\circ$ (Diaz-Rodriguez, Leroueil, & Aleman,

1992). Clays of low to medium levels of sensitivity exhibit values between $0.7 < < 0.8$, whereas

structured and sensitive soils show $0.9 < < 1$. If the intrinsic property values are not known, a default form is taken as (Ladd, 1991):

which is empirically-based on four decades of experimental lab testing by MIT on various clays world

wide. Interestingly, (2b) is also a subset of (2a) for the case where $\phi' = 27^\circ$ and $= 0.80$. Finally, if the

material is lightly-overconsolidated with $OCR < 2$, the third tier estimate is simply (Trak, et al. 1980;

Jamiołkowski, et al. 1985):

which is based in good part on corrected vane shear data from backcalculated failures of embankments,

excavations, and footings on soft clays. If more specific modes are needed, these can be assessed using constitutive relationships (e.g.,

Wroth, 1984; Ohta, et al. 1985; Kulhawy & Mayne 1990), or empirical trends based on plasticity index 0.1 1 10 1 100 Overconsolidation Ratio, OCR

D
 S S
 U
 n d
 r a
 i n
 e d
 S
 t r e
 n g
 t h
 , s u
 / $\sigma_v o$

' Amherst CVVC Atchafalaya Bangkok Bootlegger Cove Boston
 Blue Cowden Drammen Hackensack Haga Lower Chek Lok Maine
 McManus Paria Portland Portsmouth Silty Holocene Upper Chek
 Lok 40° 30' 20" ϕ 40° 20' 30" s u / $\sigma_v o$ ' $\% \sin \phi$ ' OCR Λ
 Note: $\Lambda \approx 1 - C_s / C_c \approx 0.8$ Intact clays 10 Λ

Figure 9. Experimental and CSSM variation of undrained DSS
 shear strengths with OCR. Regression: $y = 1.09 + 0.022 R^2 = 0.9695$

0.0
 0.1
 0.2
 0.3
 0.4
 0.5
 0.6

0.7

0.8

0.9

1.0 0.0 0.1 0.2 0.3 0.4 0.5 0.6 0.7 0.8 0.9 1.0 Effective
Stress, $p' * \frac{(\sigma_1' - \sigma_2') + (\sigma_2' - \sigma_3')}{3\sigma_p'}$

D

e v

i a

t o

r i c

S

t r e

s s

q *

(σ_1

σ

3) /

σ

p' Bootlegger Cove Clay Triaxial Data Assuming no
intercept $M_c = (q/p')^f = 1.10$ $M_c = 6 \sin \phi' / (3 - \sin \phi')$ $\phi' =$
 27.7° $k' = 2c' = 0.022Pc'$: $\phi' = 27.5^\circ$ and $c' = 0.011$
 $Pc' = 0.1$ 1 10 1 10 100 Overconsolidation Ratio, OCR S t r e
n g t h R a t i o , s_u / σ_{vo}' Critical State Soil
Mechanics (Modified Cam Clay) $\phi' = 27.7^\circ$ $\Lambda = 0.75$ DSS Data
CIUC Data MCC Pred CIUC MCC Pred DSS

Figure 10. Measured and predicted s_u / σ_{vo}' vs. OCR for
Bootlegger Cove Clay, Anchorage.

(e.g, Jamiolkowski et al. 1985; Ladd, 1991). For example,
the triaxial compression mode (CIUC) that

can be accommodated by most commercial laboratories is

given by:

where $M = 6 \sin \phi' / (3 - \sin \phi')$ represents the frictional parameter in Cambridge $q - p'$ space. Thus, CSSM

offers a nice framework to organize laboratory strength data in terms of the stress history of the clay,

as well as a means of predicting changes in strength should onsite construction require the addition

of new fill loading, embankments, and/or excavation. For illustrative purposes, Figure 10 shows a

set of 15 CIUC triaxial tests and 5 DSS tests on Bootlegger Cove Clay from Anchorage, Alaska in

terms of normalized undrained shear strengths vs. overconsolidation ratio (Mayne & Pearce, 2005).

The data are well-represented by CSSM equations (2a) and (3) using the triaxial-determined friction

angle $\phi' = 27.7^\circ$ and compression parameters from standard consolidation tests ($C_c = 0.24$; $C_s = 0.06$;

$= 1 - C_s / C_c = 0.75$). Note that for fissured clays (generally associated with high OCRs), the values from (2a), (2b), and (3)

should be reduced by as much as one-half because of the macrofabric of existing fractures and cracking.

For soils that are overconsolidated by mechanical unloading, a limiting OCR is reached in extension when

the lateral stress coefficient (K_0) reaches the passive value (K_p). For simple virgin loading-unloading of

“normal soils” that are not highly cemented nor structured, the K_0 value increases with OCR according to:

If the passive condition is represented by the general condition, then K_0 cannot exceed:

Generally, the value of effective cohesion intercept $c' \approx 0$, although a small finite value can be sometimes

justified as a projection from the portion of the yield surface extending above the frictional envelope. In

these cases, $c' \approx 0.02\sigma'_{p'}$ may be applicable (Mayne & Stewart, 1988; Mesri & Abdel-Ghaffar, 1993).

2.6 Hybrid SCE-CSSM model for cone penetration

Cone penetration in clays can be modeled using a number of different theories (Yu & Mitchell, 1998),

including limit plasticity (e.g., Konrad & Law, 1987), cavity expansion (Keaveny & Mitchell, 1986), and

strain path methods (Whittle & Aubeny, 1993), as well as by finite elements (Houlsby & Teh 1988) and

dislocation theory (Elsworth, 1998). Using a spherical cavity expansion solution (Vesic, 1977) with a

CSSM representation of undrained loading for triaxial compression mode, Mayne (1991, 1993) showed

that the cone tip stress could be expressed in closed-form by:

Similarly, the penetration porewater pressure at the cone shoulder ($u_b = u_2$) can also be expressed in

terms of these parameters:

Porewater pressures measured by type 1 cones (designated u_1) depend upon the actual position of the

filter element (apex, below or above midface), as shown by Chen & Mayne (1994). An additional

component representing compression beneath the tip can be simulated by an elastic stress path, such that

for a typical midface element (Mayne, Burns, & Chen 2002):

where s^* is the slope of the total stress path in Cambridge q - p' space, averaging around $\Delta q / \Delta p' = 3/4$ for

midface elements. A large database of 45 intact clays tested by paired type 1 and type 2 piezocones, or else

by special multi-element penetrometers, shows good agreement for (5) in Figure 11 using representative

values of $M = 0.92$, $\lambda = 0.80$, and $s^* = 0.75$. Data from five

fissured clays are also shown for comparative

purposes. The fissured clays are of particular interest in that positive recordings are observed at the

tip/face (u_1), whereas readings at the shoulder (u_2) are negative (Mayne, et al. 1990). Nevertheless, (8)

is seen to provide a reasonable transform from u_2 to u_1 for these fissured clays too. The sleeve friction in clays can be taken similar to a "Beta" method for calculating side friction of

piles (e.g., Finno 1989). In this notion, the CPT sleeve resistance can be expressed by:

where δ' is the interface friction between the steel penetrometer and the soil. Based on recent interface

roughness research, a representative value can be taken as $\tan \delta' / \tan \phi' = 0.4$. Using this with (4) can

provide a first-order evaluation of CPT f_s in clays. To illustrate the forward use of stress history to drive the readings of cone tip, porewater pressures,

and sleeve friction, Figure 12 shows the matched profiles obtained from the fitting of CPTu data for

Sarapuı clay in Rio de Janeiro (data from Almeida & Marques, 2003) with parametric values: initial void

ratio $e_0 = 3.0$, operational prestress $= \sigma' p = 25$ kPa, $\phi' = 29^\circ$, $\alpha = 0.7$, and operational rigidity index

$I_R = 50$. The groundwater lies 0.2 m deep and the void ratio generates an average (light) unit weight

for calculation of the total and effective overburden stresses. The overconsolidation has been caused by
Regression: Intact Clays: Δu_1 (meas) = 1.00 Δu_1 (pred) $R^2 = 0.893$ ($n = 556$) 0

500

1000

1500

2000

2500 0 500 1000 1500 2000 2500 Δu 1 Evaluated from Δu 2

M

e a

s u

r e

d

P e

n e

t r a

t i o

n

Δu

1

(k P

a) Intact Lab Silt Leda Fissured Cem/MicroFiss Linear
(Intact) Arithmetic Regressionon Intact Clays (n = 556) Δu
1MEAS = 1.00 Δu 1PRED r 2 = 0.893 10 100 1000 10000 10
100 1000 10000 Δu 1 Evaluated from Δu 2 M e a s u r e d P
e n e t r a t i o n Δu 1 (k P a) Intact Lab Silt Leda
Fissured Cem/MicroFiss Linear (Intact)

Figure 11. Measured midface u 1 porewater pressures from
shoulder u 2 readings in (a) arithmetic and (b) log-log

plots from 45 intact clays (data from 5 fissured clays also
shown).

0

1

2

3

4
5
6
7
8
9
10
11
12
13
14 0 200 400 600 800 Cone Stress q t (kPa)
D
e p
t h
(m
) SCE-CSSM qt Meas qt 05 0 1 2 3 4 5 6 7 8 9 10 11 12 13 14
0 200 400 600 Porewater Pressure u M (kPa) Meas u1 Meas u2
SCE u1 SCE u2 u0 0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 0 20 30
Sleeve Friction f s (kPa) Meas fs 05 Beta fs 0 1 2 3 4 5 6
7 8 9 10 11 12 13 14 0 5 7 Overconsolidation Ratio
Oedometers Geol. Profile 10 81 2 3 4 6

Figure 12. Generated and measured CPTu profiles in Sarapuı soft clay, Brazil. (Data from Almeida & Marques, 2003).

ageing (Almeida & Marques, 2003), yet it suffices to use an equivalent calculated $OCR = [\sigma' p / \sigma' vo + 1]$.

Then, equations (6) through (9) are adjusted to match the measured CPT readings with depth.

2.7 Hybrid SCE-CSSM model for dilatometer

Although the flat dilatometer geometry does not directly lend itself to application by cavity expansion

theory, the initial generated porewater pressure isochrones are reasonably symmetric about the blade at

early stages of penetration (Huang, et al. 1991). Prior studies have shown considerable similarity between

the initial lift-off or contact pressure (p_0) measured by the flat dilatometer test (DMT) and the measured

porewater pressures during cone penetration (Mayne & Bachus, 1989; Mayne 2006a). Also, similarities

exist between the DMT p_0 and limit pressure (P_L) from companion series of pressuremeter tests (PMT)

in the same clays (Clarke & Wroth, 1988). Using the latter as a guide, let us adopt (1) as applicable to

the DMT using p_0 to represent the limiting pressure:

We can assume the same value of rigidity index applies to the DMT as to the CPT, as both invoke

similarly large levels of straining during installation. The total horizontal stress can be calculated as

$\sigma_{ho} = \sigma'_{ho} + u_o$, using the fact that $\sigma'_{ho} = K_0 \sigma'_{vo}$. Table 1. List of clay sites with force-fitted CPTu and DMT calibrations and derived parameters. z_w Void σ'_{p} OCR Rigidity Site name Location (m) ratio e_0 (kPa) State ϕ' I R Baton Rouge Louisiana 5 0.92 850 4 to 15 28.5 $\circ$ 0.70 85 Bothkennar Scotland 1 1.50 40 1 to 2 39.0 $\circ$ 0.90 85 Cooper Marl S. Carolina 1 1.35 480 4 to 8 44.0 $\circ$ 0.90 750 Ford Center Illinois 1 0.80 55 1 to 2 28.0 $\circ$ 0.80 300 Onsøy Norway 0.5 1.75 30 1.2 to 3 32.0 $\circ$ 0.80 75 Sarapuí Brazil 0.2 3.00 25 1 to 3 29.0 $\circ$ 0.70 50 Notes: z_w = depth to groundwater table; σ'_{p} = prestress (also termed OCD by Locat, et al. 2003); OCR = overconsolidation ratio; ϕ' = effective stress friction angle; $= 1 - C_s / C_c$ = plastic volumetric strain potential; C_s = swelling index; C_c = virgin compression index; and I R = G/s u = operational rigidity index. For the expansion pressure measured by the DMT, a similar stress path concept (as used previously

for the CPT u_1 reading) can be assumed whereby:

While (11) is far from rigorous, it does in fact provide

reasonable values for the DMT material index,

$I_D = (p_1 - p_0) / (p_0 - u_0)$ that is useful in classifying soil types and consistency.

2.8 Case study applications in clays

The simple representations for stress history ($\sigma' p$, OCR) are used to generate full profiles of CPT q_t ,

f_s , u_1 , and u_2 , as well as DMT pressures p_0 and p_1 for three soft clays and two stiff overconsolidated

soils. The soft clays include the UK national test site at Bothkennar (Hight, et al. 2003), the soft clay

at Onsøy used by NGI (Lunne, et al. 2003), and a site very near the national geotechnical test site at

Northwestern University (Finno, et al. 2000). For all these sites, a representative average void ratio and

friction angle were known from lab tests on undisturbed samples taken from the site, as well as profiles

of $\sigma' p$ from oedometer and/or consolidation tests. To check on the validity to stiffer clays, the method

was also applied to two additional soils: (a) a sandy calcareous clay from Charleston, SC, known as the

Cooper Marl (Camp, et al. 2002); and (b) stiff desiccated clay from Baton Rouge, Louisiana (Chen &

Mayne, 1994). Table 1 summarizes the necessary input parameters that drive the complete analysis for

each of the sites. Forward fitting of the SCE-CSSM algorithms for the in-situ tests in these six clays

are presented in Figures 12 through 18. Brief descriptions of the clay sites, reference sources, and the

calibrated fittings to in-situ test data are given below for each of the sites. The Bothkennar clay site is located in Scotland and served as a major national test site for the British

geotechnical community (Hight, et al. 2003). The entire June 1992 issue of Geotechnique is devoted to

papers on the geology, geotechnical aspects, and lab and field testing in the soft Bothkennar clay (e.g.

Nash, et al. 1992). The subsurface conditions consist of Holocene estuarine clays of 20-m thickness or

more that have become lightly preconsolidated due to slight erosion ($\sigma'_{p} = 15$ kPa) and groundwater

fluctuations ($1 < z_w < 3.5$ m), plus additional structuration, including ageing since deposition some

8000 to 3000 years before present. Index testing shows plasticity characteristics changing with depth:

$50 < w_n < 75\%$, $25 < w_p < 35\%$; and $55 < w_L < 85\%$. The Bothkennar clay has a rather high effec

tive stress friction angle at peak of between 36° and 45° . Using representative values of $\phi' = 39^\circ$ and

$I_R = 85$, Figure 13 shows the forward fitting of the SCE-CSSM expressions for cone, piezocone, and flat

dilatometer by adopting an equivalent prestress of $\sigma'_{p} = 40$ kPa. Here, additional types of piezocone

data from u_1 (midface) and u_2 (shoulder) elements were also available (Powell, et al. 1988; Jacobs &

Coutts 1992). The soft Onsøy clay in Norway has served as an experimental test site for NGI for almost 40 years

(Lacasse & Lunne, 1982; Lunne, et al. 2003). The site consists of approximately 40 m of uniform

marine clay of Holocene age. Primary mechanisms for apparent overconsolidation include groundwater

fluctuations and ageing. Index parameters for the Onsøy clay include: $50 < w_n < 70\%$; $w_p \approx 30\%$; and

$50 < w_L < 80\%$. Using representative values of $\phi' = 32^\circ$ and $I_R = 75$, Figure 14 shows the good fitting

observed for the SCE-CSSM expressions for cone, piezocone, and flat dilatometer by adopting an

equivalent prestress of $\sigma'_{p} = 30$ kPa. 0 2 4 6 8 10 12 14

16 18 20 22 0 100 200 300 Preconsolidation Stress, σ_p (kPa) Depth (m) Geologic Profile Lab CRS Lab Oed Lab RF Eff. Overburden 0 2 4 6 8 10 12 14 16 18 20 22 0 1 4 Overconsolidation Ratio, OCR Depth (m) Geologic Profile Lab CRS Lab Oed Lab RF

0

2

4

6

8

10

12

14

16

18

20

22 0 500 1000 1500 Cone Readings q_t (kPa)

D

e p

t h

(m

) SCE-CSSM q_t Meas q_t Meas 1992 0 2 4 6 8 10 12 14 16 18 20 22 0 500 1000 Porewater Pressure u_2 (kPa) Meas u_1 Meas u_2 SCE u_1 SCE u_2 u_0 0 2 4 6 8 10 12 14 16 18 20 22 0 10 20 30 Sleeve Friction f_s (kPa) f_s (Powell 2006) Beta f_s 0 2 4 6 8 10 12 14 16 18 20 22 0 200 400 600 800 DMT Pressures (kPa) meas p_0 meas p_1 SCE p_0 SCE p_1 2 3 5 6

Figure 13. Fitted OCR, CPT, CPTU, & DMT profiles for soft Bothkennar clay (data from Hight et al. 2003).

0

2
 4
 6
 8
 10
 12
 14
 16
 18
 20
 22 0 500 1000 Cone Readings q_t (kPa)
 D
 e p
 t h
 (m
) SCE- CSSM q_t 1995 0 2 4 6 8 10 12 14 16 18 20 22 0 500
 1000 1500 Porewater Pressure u_M (kPa) Meas u_1 Meas u_2
 SCE-CSSM u_1 SCE-CSSM 0 2 4 6 8 10 12 14 16 18 20 22 0 10 20
 30 Sleeve Friction f_s (kPa) Meas f_s 05 Beta $f_s \times 7$ 0 2 4 6
 8 10 12 14 16 18 20 22 0 200 400 600 800 Dilatometer
 Pressures (kPa) DMT p_0 DMT p_1 CEE CCE+ u_0 0 2 4 6 8 10 12
 14 16 18 20 22 0 100 200 300 Preconsolidation Stress, σ_p'
 (kPa) D e p t h (m) Geologic P_c' 1973 Tubes (95 mm) 1985
 Block Samples 1985 Tubes (95 mm) 2001 Block Samples 2001
 Tubes (54 mm) Eff. Overburden Geologic P_c' 1973 Tubes (95
 mm) 1985 Block Samples 1985 Tubes (95 mm) 2001 Block
 Samples 2001 Tubes (54 mm) Eff. Overburden 0 2 4 6 8 10 12
 14 16 18 20 22 0 2 4 Overconsolidation Ratio, OCR D e p t h
 (m) 1 3 5 6

Figure 14. Fitted OCR, CPT, CPTU, and DMT profiles for soft
 Onsøy clay (data from Lunne et al. 2003). Tip Resistance 0
 5 10 15 20 25 30 0 4 8 10 12 14 q_T (MPa) D e p t h (m)
 $fcpt_1$ $fcpt_2$ $fcpt_3$ $fcpt_4$ $fcpt_5$ Sleeve Friction 0 100
 200 300 f_s (kPa) Porewater Pressure 0 500 1000 1500 2000

u 2,1 (kPa) u 1 u 2 u 0 Shear Wave Velocity 0 100 200 300
400 500 V s (m/s) SCPT Downhole (pseudointerval) Frequent
True-Interval Probe 2 6

Figure 15. Summary of SCPTU soundings and frequent V s
profiles at Ford Center, Evanston, Illinois. 0 2 4 6 8 10
12 14 16 18 20 22 0 500 1000 1500 Cone Readings q t (kPa)

D

e p

t h

(m

) SCE-CSSM qt Meas qt 02 Meas qt 05 0 2 4 6 8 10 12 14 16
18 20 22 0 500 1000 1500 Porewater Pressure u M (kPa) Meas
u1 Meas u2 SCE-CSSM u1 SCE-CSSM u2 u0 0 2 4 6 8 10 12 14 16
18 20 22 0 10 20 30 40 50 60 Sleeve Friction f s (kPa) 0 2
4 6 8 10 12 14 16 18 20 22 0 200 400 600 800 1000 1200
Dilatometer Pressures (kPa) DMT p0 DMT p1 CCE CCE + Meas fs
02 Meas fs 05 Beta fs

Figure 16. Fitted OCR, CPT, CPTU, and DMT profiles for soft
clay at Ford Center, Illinois. Soft freshwater clay
deposits underlie the Ford Design Center (Blackburn, et al.
2005; Mayne 2006a)

which is located near the national geotechnical
experimentation site (NGES) on the campus of North

western University in Evanston, Illinois (Finno, 1989;
Finno, et al. 2000). The author's CPT crew at

Georgia Tech (GT) conducted CPTUs and DMTs at both the Ford
site and nearby NGES in 2003 for the

National Science Foundation (NSF). At both sites, an upper
sand fill overlies soft clays that are lightly

overconsolidated with $1.2 < OCRs < 2$. Results from the 5
SCPTUs, one DMT, and a special frequent

interval downhole-type V s probing (Mayne 2005) are
presented in Figure 15. These soundings delineate

the soft clays between 5 and 22 m depths. The forward
fitting of CPT and DMT data are presented in

Figure 16 using a prestress $\sigma'_p = 55$ kPa, representative $\phi' = 28^\circ$ (Finno 1989), and rather moderate

high value of $I_R = 300$. To illustrate the approach applicability for overconsolidated materials, two additional case studies

from the author's files have been included here. The stiff clays at the Baton Rouge site are Pleistocene age materials extending from 5 to over 40

meters thick (Chen & Mayne, 1994). In-situ test profiles are presented in Figure 17. These deltaic clays

are fissured and desiccated. They are geologically & geographically related to the Beaumont clays of the

Houston NGES (e.g., O'Neill, 2000). Mean values ($\pm$ one S.D.) from lab index testing gave the water

content $w_n = 34 \pm 12\%$; liquid limit $w_L = 60 \pm 20\%$, and plasticity index, $PI = 33 \pm 13\%$. Seven CPTs

and one DMT sounding were performed using the cone truck at the Louisiana Transportation Research

Figure 17. Fitted OCR, DMT, CPT and CPTU profiles for stiff clay in Baton Rouge, Louisiana.

Center (LTRC) by the author under an NSF grant. The results of the piezocone type soundings were

obtained using a Fugro triple-element penetrometer that showed good quality data at the midface (u_1)

porewater pressure position, but irregular results at the shoulder (u_2) and behind sleeve (u_3) locations,

probably due to use of water as the saturating fluid rather than the now-preferred glycerine (Mayne et al.

1995). The overconsolidation difference (OCD), as termed by Locat et al. (2003), is a quasiprestress

(σ'_p) that apparently varies with depth in the stiff Baton Rouge clays. Alternatively, a constant quasi

preconsolidation stress on the order of 1 MPa gives similar results (Chen & Mayne 1994). A series of 9

triaxial tests of the CIUC type determined $c' = 0$ and $\phi' = 28.5^\circ$ for these materials. Using these values

together with $I_R = 85$ gave the profiles of OCR, q_t , u_1 , u_2 , f_s , and DMT p_0 and p_1 for this site (Figure 16).

It can be seen that the general magnitudes for CPTU and DMT readings are in line, yet the desatu

rated shoulder piezocone elements fall below the expected values, for reasons given above. Additional

“squiggly lines” and variances in the measured in-situ profiles reflect other influences, including the

macrofabric of fissures and a more complex stress history (as these are Pleistocene age materials), as

well as variable sand fractions. Nevertheless, the fitted profiles are notably within the realm of measured

resistances that are considerably higher than the four previously-shown soft clays at Bothkennar, Onsøy,

Sarapuí, and Ford sites.

The stiff Cooper Marl lies beneath Charleston, South Carolina, and serves to support most of the

large building, dock, and bridge structures in the region. This material appears as a sandy calcareous

clay of Oligocene age, with a very high calcium carbonate content between 60 to 80% (Camp, et al.

2002). Representative mean values of index parameters include: $w_n = 48\%$, $w_L = 78\%$, and $PI = 38\%$.

Typical SPT-N values in this material are in the range of 12 to 16 bpf. A representative SCPTU is given in

Mayne (2005). Separate sets of triaxial testing on undisturbed samples of the calcareous clay by various

geotechnical firms for the newly-opened Arthur Ravenel Bridge in 2005 and 15-year old Mark Clark

Bridge show consistently high effective stress friction angles $40^\circ \leq \phi' \leq 45^\circ$ for the Cooper Marl. The

material has been preconsolidated by erosion and groundwater changes, as well as added structuration

due to the calcium carbonate chemistry. Using an $OCR = 480$ kPa, representative $\phi' = 44^\circ$, and very

high $I_R = 750$, derived profiles of OCR, CPT, CPTU, and DMT resistances are given in Figure 18.

2.9 First-order OCRs from piezocone and dilatometer

The aforementioned hybrid SCE-CSSM expressions can be used to help ascertain OCR and/or OCD

profiles in clay deposits, requiring input values of effective friction angle, compressibility indices, 10 12 14 16 18 20 0 200 400 600 800 1000 Preconsolidation Stress (kPa) Depth (m) Geologic P_c' Oed. A Oed.B 10 12 14 16 18 20 0 6 10 Overconsolidation Ratio, OCR Geologic P_c' Oed. A Oed.B 10 12 14 16 18 20 0 Dilatometer Pressures (kPa) DMT p_0 DMT p_1 CCE CCE + 10 12 14 16 18 20 0 2000 4000 6000 Cone Readings q_t (kPa) Depth (m) SCE-CSSM q_t Meas q_t 10 12 14 16 18 20 0 1000 2000 3000 4000 Porewater Pressure u_M (kPa) Meas u_2 SCE-CSSM u_2 u_0 10 12 14 16 18 20 0 20 40 60 80 Sleeve Friction f_s (kPa) 1 2 3 4 5 7 8 9 1000 3000 2000 Meas f_s 05 Beta f_s

Figure 18. Fitted OCR, DMT, CPT, and CPTU profiles for stiff calcareous Cooper Marl, SC.

and rigidity indices. A simplified approach utilizes representative values (e.g., $\phi' = 30^\circ$, $\alpha = 0.80$, and

$I_R = 100$) to obtain first-order expressions for cone and piezocone penetration in clays (Mayne 2005)

that have also been validated by statistical regression analyses of databases in intact soft to firm to stiff

clays (Kulhawy & Mayne 1990):

Independent studies by Demers & Leroueil (2002) confirmed the applicability of (12a) for 22 clay sites

in eastern Canada. Similarly, expressions for first-order DMT evaluations of preconsolidation stresses in intact clays

have been developed (Mayne, 2001):

For fissured clays, the above relationships underpredict the preconsolidation stresses because the addi

tional macrofabric of cracks & fissures tends to open up during the advancement of the in-situ probe.

In contrast, in the benchmark lab test that defines σ'_p (i.e., the one-dimensional consolidation test), the

fissures continually close up during constrained compression. Thus, the coefficient terms for fissured

geomaterials in the above expressions could be as much as double those shown for intact clays (Mayne &

Bachus, 1989). 0.0 0.1 0.2 0.3 0.4 0.5 0.6 0 10 20 30 40 50
60 70 80 90 100 Plasticity Index, I_p (%) S_{uv} / σ'_p
Trend Mayne & Mitchell (1988) E Canada (Leroueil &
Jamiołkowski, 1991) Other (Leroueil & Jamiołkowski, 1991)
VST (Chandler 1988)

Figure 19. Evaluation of clay preconsolidation from vane strength and plasticity index.

2.10 Other in-situ tests in clays

It is also possible to infer σ'_p profiles in clays from other in-situ tests (SPT, VST, PMT, T-bar, and S-wave

velocity). For the vane, the relationship depends upon the clay plasticity (Mayne & Mitchell, 1988), as

shown in Figure 19.

The trend for the VST data is represented by:

For the standard penetration test (SPT), the measured N-value should be corrected to an energy efficiency

for the standard-of-practice (60% in the USA), designated N_{60} (e.g., Skempton 1986). In firm to stiff

clays which are not highly sensitive nor structured, the approximation is (Kulhawy & Mayne, 1990):

where $\sigma_{atm} = 1 \text{ atmosphere} = 1 \text{ bar} = 100 \text{ kPa} \approx 1 \text{ tsf}$.

For the pressuremeter test, a cavity expansion formulation can be implemented to give (Mayne &

Bachus, 1989):

where $I R = G/s_{uPMT}$ is the operational value from the PMT. As seen in Figure 20, this is only to be used

as a rough guide as the PMT is hardly utilized as an index test for profiling. Data are from self-boring

type PMT collected by Mayne & Kulhawy (1990). For the PMT, the value of $I R$ is generally higher than

that corresponding to the large strain $I R$ value appropriate for the penetration-type probes.

The relatively recent development of full-flow penetrometers have emerged to address the very soft

clays found offshore at the murky mudline region (Watson et al. 1998). These offer 3 advantages over

classical CPT. First, in lieu of a conventional conical point with 60-degree angle, the penetrometer can

be fitted with a larger head-piece (ball, T-bar, or plate) at the front-end to increase resolution of the

electronic load cell (i.e., generally 100-cm² area vs. standard 10-cm² with cone). On the second point,

the correction of porewater pressures on unequal areas becomes less significant (q_{Tcorr}) because the

measured force is much larger. Thirdly, since the soil is assumed to flow around the head, the tip stress

is used directly (as compared with net resistance for the CPT: $q_t - \sigma_{vo}$), thus avoiding necessity and

uncertainty in evaluation of the overburden stress at each depth. For very soft to soft clays and silts, a

similar development in the direct assessment of σ'_p can be made by compilation of a database at sites 10 100 10 100 1000 10000 1000 10000 PMT: $S_{uPMT} \ln(I R)$ (kPa) Lab
Oedometer P_C' (kPa) (kPa) Fissured $n=5$
Intact $n=29$ Trend $\sigma'_p = 0.5 s_{uPMT} \ln(IR)$

Figure 20. Trend between preconsolidation stress and PMT-calculated σ'_u in clays. T-bar Database in Clays 0 50 100

150 200 250 300 0 200 400 600 800 T-bar Resistance, q_{tbar} (kPa) Preconsolidation Stress, σ'_p (kPa) Onsoy Burswood Amherst Louiseville Gloucester Athlone Regression Regression ($n = 26$) $\sigma'_p = q_{tbar} / 2.8$
 $r^2 = 0.846$

Figure 21. Trend between preconsolidation stress and T-bar resistance in very soft clays.

with known stress history profiles. As such, T-bar penetration data from Onsoy, Norway (Lunne, et al.

2005), Gloucester, Ontario (Yafrate & DeJong 2005), Athlone, Ireland (Long & Gudjonsson 2004),

Burswood, Australia (Chung, 2005), Amherst, Massachusetts (DeJong et al. 2004), and Louiseville,

Quebec (Yafrate & DeJong, 2005) are collected in Figure 21 showing:

having a statistical coefficient of determination $r^2 = 0.846$ ($n = 26$). 10 15 20 25 30 35 40 45 0 100 200 300 400 500 600 700 800 900 1000 1100 1200 Preconsolidation Stress, $P_c' = \sigma'_p$ (kPa) Depth Below Water (meters) $0.33(q_{tnet}) - 0.53(u_2 - u_0)$ VST Vs Trend Oedometer OCD = 400kPa s_{vo}'

Figure 22. Combined in-situ and lab data for preconsolidation profile in Bootlegger Cove clay.

Finally, the results of shear wave velocity measurements (primarily downhole data) have been used to

obtain a first-order evaluation of σ'_p in clays (Mayne, Robertson, & Lunne, 1998):

where the preconsolidation stress is stated in units of kPa and shear wave velocity input in units of

meters/sec ($n = 262$; $r^2 = 0.823$). This may prove valuable when using non-invasive types of geophysics

in clay geologies, such as the surface wave measurements by SASW, CSW, and MASW.

For the above dataset, the coefficient of determination increased ($r^2 = 0.917$) if both net cone resistance

and shear wave were included, giving:

thus optimized by use of the seismic piezocone, since both parameters are determined during the same

sounding. In independent validations, Jamiolkowski & Pepe (2001) used (19) with success in Pancone

clay at the Pisa tower site.

2.11 Combinative in-situ data

In the interpretation of in-situ tests, there is always uncertainty in the application of available empirical,

analytical, and/or numerical methods to a particular clay formation. Thus, it behooves the site char

acterizer to use a combination of different readings and measurements taken at the site and compare

their results for agreement and conformity. Else, convince the client to ante up more funds to conduct

additional sampling and testing at the site. By using different test data together, the best estimate profile

of stress history can be obtained.

An example of combining in-situ test data is shown below in Figure 22 for the offshore charac

terization of the lower two facies of the Bootlegger Cove Formation (BCF) at the Port of Anchorage

expansion (Mayne & Pearce 2005). A jackup platform (SeaCore) was used for the drilling, sampling, and

in-situ testing, since high-to-low tidal variations are 10 m twice a day. The results of cone tip stress,

piezocone porewater pressures, vane shear, shear wave velocity, and lab oedometer tests can be col

lected together to assess the σ'_{p} profile in the clay. The preconsolidation stress then serves as a common

denominator for all test data interpretation in the Bootlegger Cove clays. The approach here for the

BCF clay shows the overconsolidated nature in consistent

manner with an assumed simple erosional

loss or $QCD = 400 \text{ kPa}$. Actual inspection of Fig. 22 shows a more complex stress history, with perhaps

sequential layers of different equivalent prestresses, such as $QCD = 320 \text{ kPa}$ from mudline level at 13

to 29 m, $QCD = 460 \text{ kPa}$ from 29 to 37 m, followed by $QCD = 390$ from 37 to 45 m. Below depths of

38 m, the σ_u relationship shows a higher interpretation of σ'_p than the net cone resistance value, perhaps

signifying a more sensitive facies in the lower part of the clay. If classical "clay" methods of interpreting undrained shear strengths directly from the lab and field

tests were applied instead, significant scatter plots would result, as s_u exhibits many faces and modes.

In concept, σ_p is uniquely defined as the yield point on the 1-D consolidation curve, corresponding to

the top of the yield surface in 3-D $q - p$ space. If one desires an undrained shear strength value, then

application of (2) provides this in a consistent CSSM framework for a representative DSS mode (Trak

et al. 1980; Jamiolkowski et al. 1985), else the user can buy into a constitutive soil model to provide other

required modes (e.g., CAUC, PSE), such as discussed elsewhere (Ohta et al. 1985; Wroth & Houlsby

1985; Kulhawy & Mayne 1990; Whittle 1993), or by use of empirical methods based on clay plasticity

(e.g., Ladd, 1991).

2.12 Stress-strain-strength representation for clays

The stress-strain-strength response of clays begins at the initial small-strain stiffness, represented by the

maximum shear modulus: $G_0 = G_{\max} = p' / (2s_u)$. The equivalent elastic Young's modulus is also available

from: $E = 2G(1 + \nu)$, where $\nu' = 0.2$ applies for drained

and $\nu = 0.5$ for undrained conditions. The direct simple shear (DSS) gives the best suited mode for stress-strain-strength response for three

reasons: (1) shearing is the primary mechanism involved in stability & deformation problems, as well

as soil-structure interaction situations; (2) DSS data appear much less affected by sample disturbance

than the more commonly-available triaxial compression modes (e.g., Lacasse, et al. 1985); and (3) the

DSS stress-strain-strength curve is close to the overall average of compression, shear, and extension

modes when strain compatibility considerations are made (Ladd, 1991). As such, from the DSS plots

of shear stress vs. shear strain (τ vs. γ_s), as shown in Fig. 7, we may define the following: (a) the

shear strength is the maximum shear stress (τ_{max}), (b) the slope is the shear modulus ($G = \tau/\gamma_s$), (c) the

initial stiffness is given by the small-strain shear modulus $G_0 = G_{max}$; (d) the ratio of shear modulus

to shear strength is the rigidity index ($I_R = G/\tau_{max}$); and (e) the strain at failure $\gamma_f = 1/I_R$. For drained

loading, the shear strength is given simply by $\tau_{max} = c' + \sigma'_{vo} \tan \phi'$, with $c' = 0$ applicable in most cases.

For saturated soils, the condition for undrained loading is given as $\tau_{max} = c_u = s_u$, termed the undrained

shear strength (per equation 2). At the onset of loading in the field, the soil does not yet know which

stress path will be pursued, as this depends upon the applied rate of loading relative to the permeability

of the ground. Consequently, the initial stiffness in the field is G_{max} for any mode (since u will not

develop until threshold strains are reached later). A companion to the DSS is another pure shear mode given by the torsional shear (TS) test. This has an

advantage in that the test can begin as a resonant column to directly determine the small-strain stiffness

($G_0 = G_{max}$), then proceed into the intermediate and large-strain regions to evaluate shear moduli and

shear strength. The G/G_{max} curves can be presented in terms of logarithm of shear strain (γ_s), as discussed

by Jardine et al. (1986) and Atkinson (2000), or alternatively in terms of mobilized shear stress (τ/τ_{max}),

as discussed by Tatsuoka & Shibuya (1992), Fahey & Carter (1993), and LoPresti et al. (1998). With

the local strain measurements made on the midsection of triaxial specimens, similar curves of modulus

reduction with mobilized deviator stress have been developed (e.g., Tatsuoka & Shibuya, 1992). In terms

of fitting stress-strain data, G/G_{max} vs. mobilized stress level (τ/τ_{max}) plots are visually biased towards

the intermediate to large-strain regions of the soil response. In contrast, G/G_{max} vs. $\log \gamma_s$ curves tend

to accentuate the small to intermediate-strain range. The ratio (G/G_{max} or G/G_0) is a reduction factor

to apply to the maximum shear modulus, depending on current loading conditions. The mobilized shear

stress is analogous to the reciprocal of the factor of safety ($\tau/\tau_{max} = 1/FS$). A selection of modulus reduction curves, represented by the ratio (G/G_{max}), has been collected from

monotonic TS test performed on different materials (Mayne, 2005). The results are presented in Figure 23,

where $G = \tau/\gamma =$ secant shear modulus. Additional monotonic static loading test data from special triaxial

tests with internal local strain measurements are also included. Here, an assumed constant ν has been

applied and the conversion: $E = 2G(1 + \nu)$ to permit E/E_{max} vs. q/q_{max} . Undrained tests are shown by solid

dots and drained tests are indicated by open symbols. In general, the clays were tested under undrained

loading (except Pisa), and the sands were tested under drained shearing conditions (except Kentucky

clayey sand). Similar curve trends are noted for both drainage conditions (undrained and drained) for

both clays and sands. Nonlinear representation of the stiffness has been a major focus of the recent series of conferences on

the theme Deformation Characteristics of Geomaterials. A number of different mathematical expressions 0 0.1 0.2 0.3 0.4 0.5 0.6 0.7 0.8 0.9 1 0 0.1 0.2 0.3 0.4 0.5 0.6 0.7 0.8 0.9 1 Mobilized Strength, $\tau / \tau_{\max}$ or $q/q_{\max}$ M o d u l u s R e d u c t i o n , $G / G_{\max}$ o r $E / E_{\max}$ NC S.L.B. Sand OC S.L.B. Sand Hamaoka Sand Hamaoka Sand Toyoura Sand $e = 0.67$ Toyoura Sand $e = 0.83$ Ham River Sand Ticino Sand Kentucky Clayey Sand Kaolin Kiyohoro Silty Clay Pisa Clay Fujinomori Clay Pietrafitta Clay Thanet Clay London Clay Vallericca Clay NOTES: Solid Dots=Undrained Open Dots = Drained Tests TS and TX Data

Figure 23. Modulus reduction from TS and TX data on clays and sands vs. mobilized strength.

can be adopted to produce closed-form stress-strain-strength curves (e.g., LoPresti et al. 1998; Shibuya,

et al. 2001; Santos & Correia, 2001; Tatsuoka et al. 2003; Jardine et al. 2005). For example, a simple

hyperbola requires only two parameter constants for a nonlinear stress-strain representation, however, it

can only fit one region of the strain range (small or intermediate or large), as discussed by Tatsuoka &

Shibuya (1992). Thus other expressions have been sought. One example of the derived modulus reduc

tion curves for a modified hyperbola (Fahey & Carter, 1993) is shown in Figure 24, with $f = 1$ and

$0.2 < g < 0.4$ encompassing much of the TS and TX data shown previously. The stress-strain-strength

curve can be defined by the shear strength (τ_{max}), initial shear modulus ($G_0 = G_{max}$), and adopted expo

nent ($g \approx 0.3 \pm 0.1$ for “hourglass sands” and “vanilla clays”). Then the shear stress (τ) and secant shear

modulus (G) can be determined at any load level ($\tau/\tau_{max} = 1/FS$):

Alternatively, a logarithmic function (Puzrin & Burland, 1996) can be employed with a simplified

version relying on a normalized strain parameter (x_L) in the range: $20 < x_L < 50$ for the TS data set,

where $x_L = G_{max}/G_{min}$, and $G_{min} = \tau_{max}/\gamma_f$. This has the advantage that all parameters have a recognized

physical significance. Calibration fittings for soils using the log function are discussed by Elhakim &

Mayne (2000).

An illustrative fitting for both approaches is presented for San Francisco Bay Mud, where field and lab

data are reported by Pestana, et al. (2002). Results of SCPTu sounding at the California site are given in

Figure 25. The cone tip stress, excess porewater pressures, and/or effective cone resistance can be used in

(12) to obtain the estimated profile of σ'_p and OCR, which in turn, are utilized in (2) to evaluate the DSS

mode for undrained shear strength. The stress-strain-strength curves obtained using $s_{uDSS} = 21.8$ kPa and

$G_0 = 10.3$ MPa are shown in Figure 26 for the modified hyperbola ($g = 0.35$) and log function ($x_L = 35$)

with good agreement for both cases compared with measured DSS data on undisturbed samples taken

from 7.3 m depth.

A unified constitutive model for sand and clay (Pestana & Whittle, 1999) provides theoretical rela

tionships for G/G_{max} reduction with log shear strain that are found in good comparison with cyclic data 0 0.1 0.2 0.3 0.4 0.5 0.6 0.7 0.8 0.9 1 0 0.1 0.2 0.3 0.4 0.5 0.6 0.7 0.8 0.9 1 Mobilized Strength, τ / τ_{max} M o d u l u s R e d u c t i o n , G / G_{max} g 1 g 0.6 g 0.4 g 0.3 g 0.2 Note: f 1 G G max g)1 ($\tau_{max} \tau$ •

Figure 24. Modulus reduction vs. mobilized strength using modified hyperbola (after Fahey 1998).

Figure 25. Results of SCPTU in San Francisco Bay Mud (data from Pestana et al. 2002).

from resonant column tests summarized by Vucetic & Dobry (1991). Careful studies by Shibuya, et al.

(1997) and Yamashita, et al. (2001) have shown that differences in monotonic and cyclic reduction curves

are predominantly explained by strain rate effects, thus the author suggests that monotonic curves be

used as a basis for all other curves, that afterwards, can be adjusted for strain rate and/or uppers to form

dynamic curves, as necessary. In terms of in-situ testing, the use of mechanical wave geophysics (CHT, DHT, SASW, SL, SCPT,

SDMT) is significant because the initial shear modulus can be defined. Since the SCPTu provides San Francisco Bay Mud (Pestana, et al. 2002) 0.0 0.1 0.2 0.3 0.4 0.2 4 6 7 8 9 10 11 12 Shear Strain, γ_s (%) N o r m . S h e a r S t r e s s , τ / σ'_{vo} Measure DSS (7.3 m) Modified Hyperbola Log Function 3 51

Figure 26. Measured and fitted DSS stress-strain-strength curves for San Francisco Bay Mud.

information on both the initial stiffness (G_{max}) at small-strains, as well as shear strength (τ_{max} from

s_u and/or ϕ') at large-strains, the difficult question is how to determine stiffnesses at intermediate

strain levels on a site-specific basis, without assuming values. One prospect is the cone pressuremeter

(Schnaid & Houlsby, 1992; Ghionna, et al. 1995) with a

geophone setup to produce a seismic piezocone

pressuremeter (SCPMTu). Another approach would be the utilization of paired side-by-side in-situ tests,

such as companion sets of SCPTu and DMT (Tanaka & Tanaka, 1998), or the commercially-available

SDMT (e.g., Foti, et al. 2006).

2.13 Strain rate effects

Soils exhibit strain rate effects and these can be addressed by carefully-controlled lab tests (Tat-suoka,

et al. 1997; Shibuya et al. 1997). Commonly, the triaxial test has been utilized to investigate effects of

strain rate effects on undrained stress-strain behavior, with stiffness and strength both increasing with

rate of loading (e.g., Sheahan et al. 1996). Yet, considerable research has also shown that consolidation

results (both e - $\log \sigma'_{v}$ curves and yield stress, σ'_{p}) shift up with increasing strain rate, particularly in clays

(Leroueil & Hight 2003). Consequently, the results of in-situ tests in the field are affected by strain rate.

Of additional complication in the field is that drainage can occur during testing.

For vane shear tests, data were compiled by Peuchen & Mayne (2006) from 27 field sites and 7 lab

series on clays. Two common means to show rate effects in soils include (1) time-to-failure, t_f , and

(2) power function format (Soga & Mitchell, 1996). For the VST data, Figure 27 shows the effects of

time-to-failure on normalized undrained shear strength (s_{uv}/s_{uv0}). These appear fairly consistent over a

wide range of loading conditions, particularly from 0.01 minutes to 10,000 minutes. Here, the strain rate

effect on undrained vane strength may be seen to increase on the order of 10 to 14% per log cycle which

is comparable to that observed for laboratory test data (e.g., Kulhawy & Mayne, 1990; Leroueil & Hight,

2003). At very fast rates ($t_f < 0.01$ min), apparent dynamic effects may also occur, although those data

are biased towards the small lab size equipment and use of remolded and synthetic clays.

Using the alternate plotting of normalized vane strength vs. rotation rate, the data are presented in

Figure 28. The power law format shows that the exponent term is generally between 0.05 and 0.10, in

agreement with lab triaxial and consolidation data (Soga & Mitchell, 1996).

For laboratory triaxial testing, truly undrained conditions can be maintained by locking off the drainage

ports. In this case, as the shearing rates decrease to very slow values, the undrained shear strength will

level off to a minimum value. Thus, in lieu of the semi-log function depicted in Figure 27 to represent

strain rate effects on s_u , Randolph (2004) suggested an hyperbolic sine function that captures this facet.

In the field, if the rate of loading decreases significantly, then partial to full drainage can occur during

in-situ testing, since there is no control on drainage. Lunne et al. (1997) present a table of CPT studies that

look at rate effects and these can be consulted to further investigate rate effects concerning penetration Vane Strain Rate Study 0.0 0.5 1.0 1.5 2.0 2.5 0.0001 0.001 0.01 0.1 1 10 100 1000 10000 100000 Time to Failure, t_f (minutes) Normalized Vane Strength, s_u / s_{u0} Aby Askim Backebol Bentonite Kaolin 2 Bentonite Kaolin 3 Brent Knoll Bromma Ellingsrud Goteberg Gloucester Grangemouth Gullbergsvass Kalix Korpika Littorina Lokalahti Matagami Onsoy Note: In general, a different time to failure used for reference s_{u0} value for each clay $\alpha R \approx 0.10 \alpha R \approx 0.14 t_f t_f \theta s_{u0} s_u \approx 1 \approx \alpha \cdot \log \text{ standard ref rate } \approx \approx \approx \approx$

6

8

10

12

14

16 0 2 3 Tip Stress, q_t (MPa)

D

e p

t h

(m

) 0 20 40 60 80 Sleeve f_s (kPa) 0.2 0.0 0.2 0.4 0.6

Porewater u_b (MPa) 0 100 200 300 400 Shear Wave, V_s

(m/s) 10 100 1000 10000 Time Rate, t 50 (s) 541

Figure 29. Seismic piezocone test with dissipations (SCPT_u) at the Amherst soft clay test site.

Since piezocone tests (CPT_u) directly measure penetration porewater pressures, the practical emphasis

is to utilize dissipation readings to evaluate c_v in soils, although procedures have also been developed

for PMT (Fahey & Goh 1995; Clarke & Gambin 1998) and DMT (Robertson et al. 1998; Marchetti et al.

2001). The most popular CPT_u method at present is the strain path method (SPM) solution of Houlsby &

Teh (1988), although other available procedures are discussed by Jamiolkowski et al. (1985), Senneset

et al. (1988, 1989), Danziger et al. (1997), Burns & Mayne (1998, 2002), and others.

In terms of calibrating an approach, a fairly comprehensive study between lab c_v values and piezocone

c_h values in clays and silts was reported by Robertson, et al. (1992). Assumptions were made between

the ratio of horizontal to vertical permeability to address

possible issues of anisotropy during interpre

tation. The study compared laboratory-determined results with the SPM solution (Teh & Houlsby 1991)

using data from type 1 piezocones (22 sites) and type 2 piezocones (23 sites), as well as 8 sites where

backcalculated field values of c_{vh} were obtained from full-scale loadings.

With the SPM approach in practice, it is common to use only the measured time to reach 50%

consolidation, designated t_{50} . As such, if dissipation tests are carried out at select depth intervals during

field testing, a fairly optimized data collection is achieved by the SCPT $\bar{u}$ since five measurements of

soil behavior are captured in that single sounding: q_t , f_s , u_b , t_{50} , and V_s . The results of a (composite)

SCPT $\bar{u}$ in the soft varved clays at the NGES in Amherst, Massachusetts are depicted in Figure 29. Here

the results of a GT sounding are augmented with data from a separate series of dissipations conducted

by DeGroot & Lutenecker (1994).

In lieu of the focus on a single point corresponding to 50% degree of consolidation (U_{50}), other degrees

of dissipation can be considered, or even better, an entire range of consolidation data points starting from

penetration through the entire testing time. As such, Houlsby & Teh (1988) provided time factors for

a range of porewater pressure dissipations. The degree of excess porewater pressure dissipation can be

defined by: $U^* = \bar{u}/\bar{u}_i$, where $\bar{u}_i$ = initial value during penetration. The modified time factor T^* is

defined by:

where t = corresponding measured time during dissipation and a = probe radius. The SPM solutions

between U^* and T^* for midface u_1 and shoulder u_2 piezo-elements are shown in Figures 30 and 31,

respectively. These can be conveniently represented using approximate algorithms as shown, thus offering

a means to implement matching data on a spreadsheet. 0.0
0.1 0.2 0.3 0.4 0.5 0.6 0.7 0.8 0.9 1.0 0.001 0.01 0.1 1 10
100 Modified Time Factor, $T^* U_1^* = \text{Normalized } \Delta u_1 / \Delta u_1$ i Strain path Approximation $u_1 U_1^* \approx (1 + 27T^*)^{-0.4}$ -0.4 -0.06 I R a 2 . c v h . t T^* =

Figure 30. Dissipation response for type 1 midface piezo-elements (SPM solution of Teh & Houlsby 1991). 0.0
0.1 0.2 0.3 0.4 0.5 0.6 0.7 0.8 0.9 1.0 0.001 0.01 0.1 1 10
100 Modified Time Factor, $T^* U_2^* = \text{Normalized } \Delta u_2 / \Delta u_2$ i Strain path Approximation $u_2 U_2^* \approx (0.85 + 10T^*)^{-0.45}$ -0.45 -0.08 I R a 2 c v h . t T^* = .

Figure 31. Dissipation response for type 2 shoulder piezo-elements (SPM solution of Teh & Houlsby 1991). To use the method, the following procedures are suggested:

- Assume a range of values for T^* starting from 10^{-5} to say 100. Note: Choose intermediate log values at the 1's, 2's, and 5's places.
- Calculate the corresponding U^* dissipations per the approximate expressions given in Figures 30 or 31, as appropriate.
- Calculate the actual time by rearrangement of (23) to obtain: $t = T^* a^2 I^{0.5} R / c v h$. Plot U^* vs. t .
- Conduct parametric analyses by varying $c v h$ and/or $I R$ until a fitting with the measured data is achieved.

For the $I R$ value, a value may have already been attained from the fitting of penetration readings. With the

aforementioned procedure, the results of dissipation tests from the Bothkennar soft clay site have been Data: Jacob & Coutts (1992) Bothkennar depth $z = 12\text{m}$ 0.0 0.2 0.4 0.6 0.8 1.0 1 10 100 1000 10000 100000 Time (seconds) Measured u_2 Measured u_1 SPM u_1 SPM u_2 10-cm 2 CPTUs Rigidity: $I R = 85 \text{ c v h} = 0.162 \text{ cm}^2 / \text{min}$ N o r m a l i z e d $D u / D u_i$

Figure 32. Forward SPM predictions and measured dissipations at Bothkennar site, UK. Data: Danzinger et al.

(1997) Sarapui Clay: $z = 8.2 \text{ m}$ 0.0 0.2 0.4 0.6 0.8 1.0 1
 10 100 1000 10000 100000 Time (seconds) Measured u_1
 Measured u_2 SPM u_1 SPM u_2 5-cm 2 Cone Rigidit y IR = 50 c
 $v_h = 0.122 \text{ cm}^2 / \text{min}$ N o r m a l i z e d D u / D u i

Figure 33. Forward SPM predictions and measured
 dissipations at Sarapuı site, Rio.

forward fitted using the same corresponding value of
 rigidity index given in Table 1 (I R = 85) and used

in Figure 13. Different types of 10-cm 2 cones were used (a
 $= 1.78 \text{ cm}$). The value of $c v_h$ used to drive

the analysis was chosen from full-scale footing load test
 performance reported by Jardine et al. (1995).

Both short-term and long-term footing behavior was
 monitored and a backfigured $c v_h = 8.5 \text{ m}^2 / \text{year}$

($0.0027 \text{ cm}^2 / \text{s}$) was determined. Figure 32 presents the
 dissipation fitting for both type 1 and 2 piezo

dissipation data reported by Jacob & Coutts (1992) at a
 depth of 12 m at the site. Overall, there is good

agreement between the SPM procedures and measured piezocone
 data.

A similar set of calculations have been carried out for the
 soft Sarapuı clay in Brazil. Data from 5-cm 2

quad-element piezocone dissipation tests have been reported
 by Danzinger et al. (1997) and Schnaid

et al. (1997). The I R = 50 from Table 1 has been used and
 a backfigured field value from embankment

monitoring gave $c v_h = 0.122 / \text{cm}^2 / \text{min}$ (Robertson, et al.
 1992). The SPM predictions are shown for

midface and shoulder elements in Figure 33, indicating some
 underpredictions in rate of decay. A better

match is obtained with $c v_h = 0.2 / \text{cm}^2 / \text{min}$. Vane Data
 Affected by Partial Drainage at Slow Rates 0.8 0.9 1.0 1.1
 1.2 1.3 0.01 0.1 10 100 100 0 Rotation Rate, $d\theta/dt$
 (deg/sec) N o r m a l i z e d V a n e S t r e n g t h , s
 $v / s v_0$ Field Ref Rate Lab Ref Rate Saint Louis NC Kaolin
 Saint Alban OC Kaolin Undrained Response

Figure 34. Threshold velocity on undrained vane response an onset of partial drainage.

2.15 Partial drainage considerations

If in-situ tests in fine-grained soils are conducted slower than standard rates, a partially-drained or “semi

undrained” condition may prevail whereby dissipation occurs during the penetration or probing. This

may actually be the common condition in silty soils because of the relative fast rates of in-situ testing

relative to the moderate to high permeability of these materials. Randolph (2004) has suggested special series of “twitch tests” in clays whereby a variable rate of

testing by the penetration probe provides data on both strain rate effects and partial drainage. His work

has involved piezocones and full-flow probes, such as the T-bar. A dimensionless velocity term can be

defined by:

where v = velocity and d = probe diameter. The dimensionless velocity V helps to quantify the regions

that are governed by (a) undrained behavior (and strain rate), (b) semi-drained, and (c) fully-drained

behavior. This would appear a valuable means to fit within the CSSM framework and allow a ratio

nal assessment of intermediate stress paths occurring between fully undrained and fully drained. The

demarcation between the undrained region for VST response and partially-drained behavior can be seen

for field test data from Canada and lab series on kaolinitic clay using a miniature vane in Figure 34

(Peuchen & Mayne 2006). In the above discussions of dissipation and partial drainage, monotonic decay of induced porewater

pressures has been addressed. Yet, in many overconsolidated materials, a dilatory response can occur

(e.g., Sully & Campanella, 1994; Burns & Mayne, 1998). Also, an incompatibility exists for the examples

shown as the penetration and porewater data were represented by cavity expansion but the monotonic

porewater decay addressed using strain path. What is needed is a consistent framework in SCE and/or

SPM, else in finite elements, finite differences, and/or discrete elements.

3 IN-SITU TESTS IN SANDS

Available methods for evaluating the mechanical properties of sands have developed from an assortment

of empirical-statistical, analytical, theoretical, and numerical simulation approaches. Their validity has

mostly been established on the basis of reconstituted samples tested in the lab, on either small-size

triaxial specimens ($25 < D < 75$ mm) or large calibration chamber tests (CCT with $0.9 < D < 2.5$ m),

or combination of both. The CCTs allow for the complete insertion of the in-situ device. However,

the calibration chamber results must be corrected for the limited size chambers, since flexible-walled

CCTs will under-register the measured penetration resistances compared to far-field conditions, whilst

rigid-walled CCTs will over-register the readings.

Of recent vintage, special freezing methods have been developed to obtain field undisturbed samples

of sands from the same medium as the in-situ testing locations (e.g., Hoffman et al. 1996). The expensive

and time-consuming process requires a slow moving unidirectional freezing front. This is done so that

the sand fabric and natural structure are not destroyed

during the transformation of water to ice at $T < 0^\circ\text{C}$

that is accompanied by a volumetric increase of around 8.5%.

In this section, a review of CPT interpretative methods derived from corrected CCTs for clean quartz

sands and siliceous sands (comparable parts of quartz & feldspar particles) is given, with a few well

chosen case study examples, followed by a short supplement of SPT-based methods. Then, a special

dataset based on undisturbed sands is presented to evaluate select interpretative procedures. The emphasis

in this paper is on CPT, SPT, and V_s methods, supplemented with DMT and PMT data where available.

3.1 Cone penetration in clean sands

Using a large dataset of 702 CCTs conducted on 26 different clean quartz to siliceous sands with appro

priate boundary size corrections applied to the measured CPT tip resistances, the derived relationship

for obtaining the effective stress friction angle was (Kulhawy & Mayne 1990):

where $q_{t1} = (q_{t1} / \sigma_{atm}) / (\sigma'_{vo} / \sigma_{atm})^{0.5}$ is the stress-normalized cone tip stress and $\sigma_{atm} = 1 \text{ bar} = 100 \text{ kPa}$. The

normalization to square root of effective overburden stress likely helps to account for sand compressibility

and grain crushing effects, to some degree.

The effective lateral stress applied in chamber tests affects the cone tip stress, moreso than the effective

vertical stress. Thus, the applied consolidation state used in the CCTs is given by both the lateral stress

coefficient ($K_0 \approx K_c = \sigma'_{hc} / \sigma'_{vc}$) and overconsolidation ratio (OCR) that has been related to the measured

q_{t1} values via statistical analyses of the CCT data (Mayne

2005):

The use of the above expression is rather limited, since the field evaluation of stress history of natural

sands is often quite elusive. However, if an apriori relationship between K_0 and OCR can be established,

the procedure can move forward. For instance, a methodology based on a combining of (4) with (26) is

of the form (Mayne, 2001):

where the apparent preconsolidation stress of the sand can be calculated from:

3.2 CPT case study examples

Two case studies can be used to illustrate the evaluation of strength and stress history of natural sands:

(1) nearly NC Po River Sand, Italy (Ghionna et al. 1995); and (2) OC glacial sand near Stockholm,

Sweden (Dahlberg 1974). In both cases, the stress history is relatively well-known based on geologic

setting and rather extensive lab & field testing which have been carried out.

The Po River Sand is a relatively thick deposit of clean to slightly silty sand that has been subjected to a

great number of in-situ tests, including SPT, CPT, CPMT, PMT, SBP, DMT, and V_s (Jamiolkowski, et al.

1985; Bruzzi, et al. 1986; Ghionna et al. 1995). The sand is lightly overconsolidated due to groundwater

fluctuations and ageing. Special sampling of occasional clay lenses of geologically-related materials

were captured and tested in oedometers to quantify the stress history. Additional data on the stress

state (i.e., K_0) was obtained from the independent pressuremeter testing (SBPMT). Figure 35 shows

the application of the aforementioned relationships in evaluating the respective profiles of measured q at $z = 0, 5, 10$

15 20 25 0 100 200 300 Tip Stress, q_t (atm)

D

e p

t h

(m

) 0 5 10 15 20 25 0 1 2 3 4 5 OCR CPTU Oedometer 0 5 10 15
20 25 25 30 35 40 45 50 Friction Angle, ϕ' (deg) CPT: C&R
1983 CPT: K&M 1990 Self-BoringPMT Full-Displacement PMT 0
5 10 15 20 25 0 50 100 150 200 Lateral Stress, σ'_{ho} (kPa)
SBP CPMT CPTU svo'

Figure 35. Profiles of CPT q_t , OCR, ϕ' , and σ'_{ho} for
Po River Sand (data from Ghionna et al. 1995). 0 1 2 3 4 5
6 7 8 30 35 40 45 50 55 Friction Angle, ϕ' (deg) D e p t h
(m e t e r s) 6 0 1 2 3 4 5 6 7 8 0 5 10 15 20 25
Overconsolidation Ratio, OCR Excavation CPT 12 CPT 03 CPT
11 CPT 06 0 1 2 3 4 5 6 7 8 0 1 2 3 4 5 6 7 8
Preconsolidation σ'_p (kg/cm²) CPT 12 CPT 03 CPT 11
CPT 06 SPLT-J3 SPLT-J2 SPLT-J1 Svo' Excavation Pc' 0 1 2 3
4 5 6 7 8 0 1 2 3 4 Lateral Stress Ratio, K_0 CPT 12 CPT
03 CPT 11 CPT 06 PMT Triaxial Data CPT Data

Figure 36. CPT-interpreted profiles of ϕ' , OCR, and K_0
in Stockholm Sand (data from Dahlberg 1974).

and interpreted ϕ' , OCR, and $\sigma'_{ho} = K_0 \cdot \sigma'_{vo}$ in Po
River sand. Reasonable values of $1.5 < OCR < 2.5$ are

obtained from the CPT data relative to the few oedometric
results available. Results from self-boring

type (SBP) and full-displacement type (CPMT) pressuremeter
testing have been used to independently

assess the effective ϕ' and K_0 conditions, where again
the CPT values are seen to be in general accord. For the
Stockholm Sand, a Holocene deposit of clean glacial
medium-coarse sand was quarried for use

in construction, having an initial 24 m thickness overlying
bedrock. After the upper 16 m was removed,

series of in-situ testing (SPT, CPT, PMT, SPLT) were
performed, in addition to special balloon density

tests in trenches. Groundwater lies at the base of the sand just above bedrock. Index parameters of

the sand include: mean grain size ($0.7 < D_{50} < 1.1$ mm); uniformity coefficient ($2.2 < UC < 3$), mean

density $\rho_T = 1.67$ g/cc, and average $D_R \approx 60\%$. Results of the screw plate load tests (SPLT) were used

by Dahlberg (1974) to interpret the preconsolidation stresses in the sand, very comparable to the known

geologic values from mechanical overburden removal ($OCR = \sigma'_v$). Results of the lift-off pressures

from PMTs gave corresponding K_0 values in the sand. Using results of 4 Borros electric CPTs at the

site, Figure 45 shows the derived profiles of ϕ' , OCR, and σ'_p in Stockholm sand. The CPT-interpreted ϕ'

agrees well with triaxial tests on reconstituted samples. The stress history from CPT is consistent with

the OCR and SPLT data, with additional support on K_0 given by PMT results.

3.3 SPT penetration in clean sands

For the standard penetration test (SPT), empirical methods have been produced for assessing the strength

and stress history of clean quartz to siliceous sands. Using results based on triaxial tests on frozen sand

specimens, the effective stress friction angle can be obtained from the energy-corrected and stress

normalized N-value from the SPT (Hatanaka & Uchida 1996; Mayne et al. 2002): $0.1 \leq N_{60} \leq 100$ Energy-Corrected SPT N_{60} (bpf) Preconsolidation σ'_p (bars)

Figure 37. Apparent preconsolidation stress vs. N_{60} for soils (after Mayne 1992). $0 \leq N_{60} \leq 400$. Preconsolidation Stress, σ'_p (kPa)

D

e p

t h

(m

e t e

r s

) qt-based DMT Dilly Vs - SCPT Vs (CHT-2-1) Svo 0.5(po-uo)
SPT N-value 0 1 2 3 4 5 6 7 0 1 2 3 4 5 6 7 8 9 10 11 12
Overconsolidation Ratio, OCR D e p t h (m e t e r s)
qt-based DMT Dilly Vs - SCPT Vs (CHT-2-1) SPT-N Alt DMT

Figure 38. Combinative in-situ test interpretations of stress history at College Station sand site, Texas.

The apparent preconsolidation stress in clean sands can be estimated from the relationship given in

Figure 37. This can be combined with (15) in generalized form (Mayne, 1992):

where $m = 0.6$ for clean quartzitic sands, 0.8 for silty sands to sandy silts (e.g., Piedmont), and $m = 1.0$

for intact “vanilla” clays to clayey silts.

3.4 Application case study to TAMU sand

A US national geotechnical experimentation site (NGES) has been established in Eocene age sand near

Texas A&M University (Briaud 2000). Here the 10-m thick sand is relatively clean in the upper 4 or 5

meters, becoming slightly to somewhat more silty and clayey with depth. Extensive series on in-situ tests

have been carried out at the site, including SPT (with and without energy measurements), CPTU, SCPT,

DMT, PMT, and CHT, as well as full-scale load tests on pilings and footings (Briaud & Gibbens, 1994).

The CPT evaluation gives an operational friction angle $\phi' = 39^\circ$ in the sands (Mayne 1994). The combined

use of the SPT and CPT procedures for assessing profiles of preconsolidation stress and OCR at the NGES

sand site located in College Station, Texas are shown in Figure 38. Both the SPT and CPT-based profiles

are in general agreement with each other. Also shown is an evaluation using shear wave velocity mea-

surements from the site (both DHT by SCPT and CHT), based on a generalized approach (Mayne 2005):

Available DMT data from the site was input into the DILLY software program and these profiles are

also presented, along with values from (13). Unfortunately, no real known OCR profile of this sand is

documented, so that validity of these results cannot be checked. It is therefore apparent that significant

research is needed in establishing the calibration and documentation of natural sand sites, particularly

with respect to stress history considerations.

3.5 CE and CSSM frameworks for sands

Cavity expansion theory can be applied to cone penetration in sands since it represents an inverse problem

of pile end bearing capacity (Vesic', 1972, 1977). Sand parameters include the effective friction angle

(ϕ'), effective cohesion intercept (c'), rigidity index (I_R), and magnitude of induced volumetric strain

($\Delta V/V_0$). In lieu of the latter parameter, a formulation by Carter et al. (1986) develop a CE solution in

terms of the dilatancy angle (ψ'). For the case of $c' = 0$ for these approaches, the normalized tip stress

term is q_t / σ'_{vo} and therefore requires additional information in order to assess I_R and either ($\Delta V/V_0$) or

ψ' (Mitchell & Keaveny, 1986; Yu 2004). Recently, Mayne (2006) showed that the operational rigidity

index, defined as $I_{RR} = 1/[(1/I_R) + (\Delta V/V_0)]$, may be related to the normalized shear modulus (G_0 / σ'_{vo}). Critical-state concepts have been used to organize sand

strength data together include the effects of

state (relative density), confining stress level, mode, and dilatancy, as well as differences in mineralogy

(Bolton, 1986). In this regard, the method is hindered somewhat because the penetration test data

(i.e., SPT, CPT) are used solely to evaluate the in-place relative density (D_R) while values of the fitting

parameters (Q , R , and baseline ϕ'_{cs}) are often assumed (Jamiolkowski, et al. 2001). The advantage is that

the CSSM framework does allow variants and adjustments to sands of differing origins and constituencies.

Using the Bolton CSSM relationship for peak ϕ' in sands, Salgado et al. (1997a, 1997b, 1998) developed

a numerical code (CONPOINT) that utilizes cavity expansion theory and initial shear modulus to model

cone penetration in sands. This has been extended now to address silty sands within a similar framework

(Salgado et al. 2000; Lee et al. 2004). Been & Jefferies (1985) present an alternate CSSM framework to represent sand behavior. They

define the sand state parameter (λ) as the vertical difference in void ratio between the initial in-place

condition and the critical state line (CSL), termed steady state line ($\lambda = e_0 - e_{cs}$). In this light, Been

et al. (1986, 1987) were able to relate CPT resistances to describe sand response ranging from loose

contractive to dense dilatant behavior, as well as undrained modes. Series of laboratory triaxial shear

tests are needed to define λ (and κ), yet recent efforts by Cho et al. (2006) have related the parameters to

simple index tests. The state parameter method has since been implemented into a modeling framework

for CPT (Shuttle & Jefferies, 1998). Of particular interest herein, it is possible to show that is actually

just another means to express stress history, and thus related to OCR. In fact, Been et al. (1988) show:

where OCR $p =$ overconsolidation ratio in Cambridge $q - p'$ space, $= 1 - \kappa/\lambda$, $\lambda = C_c / \ln(10) =$ compression index, and $\kappa \approx C_s / \ln(10) =$ swelling index, with latter two slopes defined in $e - \ln p'$ space

(although some confusion here as original reference is defined in terms of log base 10). The state parameter

method has since been implemented into a framework for CPT interpretation (Shuttle & Jefferies,

1998) that involves a CSSM constitutive soil model (NorSand) that appears to be quite versatile (Jefferies

& Shuttle, 2005).

3.6 Undisturbed sand database

Conventional approaches to cross-linking of lab-determined engineering parameters and in-situ field

tests have relied upon reconstituted samples, specimens, and chamber deposits. For reconstitution, a

number of preparation methods are available, including: vibration, compaction, air and water pluviation,

moist tamping, and slurring. The primary assumption is that if the sand is replaced at the same initial

density and void ratio in the laboratory, then this will give comparable behavior to the same natural sand

that exists in the field. This approach is depicted in Figure 39. It is further assumed that our present

practices of assessing the in-place relative density (D_R), unit weight (γ_T), and/or void ratio (e_0) of that

Figure 39. Procedures depicting undisturbed vs. reconstituted methods for calibrating in-situ tests in sands.

sand from in-situ test methods (SPT, CPT, DMT) is relatively reliable. Neither of these assumptions is

well-supported, as shown by recent comparisons from triaxial tests on the same sand tested in undisturbed

states as well as artificially prepared sand samples by different reconstitutive measures (e.g., Hoeg, et al.

2000; Vaid & Sivathayalan 2000; Jamiolkowski 2001). Available methods to assess D R from in-situ tests

do not appear especially reliable, particularly using penetration type tests including SPT and CPT (e.g.,

Wride et al. 2000) and DMT (e.g., Jamiolkowski, et al. 2001).

To further check on the validity of these major assumptions, a new elite database on undisturbed clean

quartz and siliceous sands was created. A total of 13 sands that were sampled using special techniques

(primarily expensive freezing technology) was collected (Mayne 2006b).

The sands are located in Canada (6 sands), Japan (4 sands), Norway (1 site), China (1 site), and Italy

(1 site), as listed in Tables 2 and 3 with pertinent index parameters and reference sources of the data.

Notably, all four of the Japanese sands (Mimura 2003), two Canadian sands (Robertson et al. 2000),

and the Holmen, Norway (Lunne et al. 2003) were discussed at the Singapore workshop. In general, the

sands can be considered as clean to slightly dirty sands of quartz, feldspar, and/or other rock mineralogy,

excepting two of the Canadian sands derived from mining operations that had more unusual constituents

of clay and other mineralogies.

In terms of grain size distributions, the materials include 10 fine sands, 4 medium sands, and

one coarse sand (Italy). The sands from Canada were dirty having fines contents: $5 < FC < 15\%$,

whereas the other sands were all relatively clean with FC < 4%. Mean values of index parameters (with

plus and minus one standard deviation) of these sands indicated: specific gravity ($G_s = 2.66 \pm 0.03$),

finer content ($FC = 4.36 \pm 4.49$), particle size ($D_{50} = 0.35 \pm 0.23$ mm) ¹, and uniformity coefficient

($UC = D_{60} / D_{10} = 2.80 \pm 1.19$). The mean grain size statistics would be better represented by a log normal

function.

At all sites, results from SPT and CPT were available, as well as downhole V_s measurements (except

the China site). A summary of mean values of the normalized SPT (N_{10}) ₆₀, normalized CPT results

¹ Note: Coarse sand from Italy with $D_{50} = 2$ mm not included. If included, then $D_{50} = 0.46 \pm 0.48$ mm.

Table 2. List of undisturbed sands, reference sources of data, origin, and sample type. Country Sampling

Sand name	location	Reference source	Type	sand method
Edo River	Japan	Yamashita et al. (2003); Mimura (2003)	Natural	Alluvial Frozen
Gioia Tauro	Italy	Ghionna & Porcino (2003, 2006)	Natural	Coarse Frozen Porcino & Ghionna (2004) Sand
Highmont	Canada	Wride & Robertson (1999);	Tailings	Frozen
Dam	Robertson et al. (2000)			
Holmen	Norway	Lunne, et al. (1986); Lunne, Natural	Alluvial	Tube Long, & Forsberg (2003) (Frozen)
J-pit	Canada	Robertson, Wride, et al. (2000);	Hydraulic	Fill Frozen Wride & Robertson (1999)
Kidd	Canada	Robertson, Wride, et al. (2000)	Natural	Alluvial Frozen Wride & Robertson (1999)
Kowloon	China	Lee, Shen, Leung, Mitchell (1999)	Hydraulic	Fill Mazier Tube

LL Dam Canada Robertson, Wride, et al. (2000) Tailings
Frozen

Massey Canada Robertson, Wride, et al. (2000) Natural
Alluvial Frozen

Mildred Canada Robertson, Wride, et al. (2000); Hydraulic
Fill Frozen

Lake Wride & Robertson (1999)

Natori River Japan Matsuo & Tsutsumi (1998); Mimura (2003)
Natural Alluvial Frozen

Tone River Japan Matsuo & Tsutsumi (1998) Mimura (2003)
Natural Alluvial Frozen

Yodo R. (8) Japan Mimura (2003) Natural Alluvial Frozen

Yodo R.(10) Japan Mimura (2003) Natural Alluvial Frozen

Yodo R.(12) Japan Mimura (2003) Natural Alluvial Frozen

Table 3. Undisturbed sands, mean depths, index parameters,
and overburden stress levels. Depth GWL D 50 σ'_{vo}

Sand name z (m) (m) PF (%) (mm) G s e o e max e min D R (%)
(atm)

Edo 3.85 2.1 0.42 0.29 2.68 1.043 1.227 0.812 44.3 0.51

Gioia Tauro 3 1.5 0.66 2 2.69 0.589 0.690 0.450 42.0 0.42

Highmont 10 4 10 0.25 2.66 0.825 1.015 0.507 37.4 1.38

Holmen 10 1 2 0.55 2.71 0.724 0.840 0.460 30.5 1.10

J-pit 5 0.5 15 0.17 2.62 0.762 0.986 0.461 42.7 0.55

Kidd 14.5 1.5 <5 0.2 2.72 0.981 1.100 0.700 29.8 1.60

Kowloon 11.8 9 1 0.72 2.63 0.492 0.634 0.328 46.5 1.80

LL Dam 8 2.1 8 0.2 2.66 0.849 1.055 0.544 40.3 1.00

Massey 10.5 1.5 <5 0.2 2.68 0.970 1.100 0.700 32.5 1.20

Mildred L. 32 21 10 0.16 2.66 0.768 0.958 0.522 43.6 5.16

Natori 8.25 2.1 0.23 0.22 2.65 0.857 1.167 0.765 77.2 0.87

Tone 7.3 1.4 3.78 0.18 2.68 0.947 1.330 0.775 69.1 0.84

Yodo 8.1 2.1 1.9 0.32 2.64 0.820 1.054 0.665 60.2 1.02

Yodo 10.9 2.1 0.27 0.82 2.63 0.720 0.883 0.569 51.9 1.23

Yodo 12.7 2.1 2.1 0.62 2.63 0.790 0.921 0.567 37.0 1.43

(Q , F , R_f , q_{t1} , and σ'_{vo}), and normalized shear wave velocity (V_{s1}) is presented in Table 4, where

$(N_{10})_{60} = N_{60} / (\sigma'_{vo} / \sigma_{atm})^{0.5}$, $Q = (q_t - \sigma'_{vo}) / \sigma'_{vo}$, $F = f_s / (q_t - \sigma'_{vo})$, $R_f = f_s / q_t$ (%), $q_{t1} = q_t / (\sigma'_{vo} \cdot \sigma_{atm})^{0.5}$, and

$V_{s1} = V_s / (\sigma'_{vo} / \sigma_{atm})^{0.25}$. An initial check on the data is made by cross-linking the in-situ measurements using prior-available

relationships. Figure 40 shows a plot of cone resistance versus the energy-corrected N-values for the

sands. A common assumption is that the direct ratio of cone tip resistance (bars) to N-value (bpf) is in the

range of $4 \leq q_t / N_{60} \leq 5$ for such materials (e.g., Schmertmann, 1978). For the 15 datapoints considered,

the ratio averages $q_t / N_{60} = 4.38$, thus within the expected range. The specific ratio q_t / N_{60} has been shown

to partially depend upon the mean grain size of the soil (D_{50}), as well as the percent fines content, FC

(e.g., Robertson & Campanella 1983; Kulhawy & Mayne 1990). However, it appears this ratio decreases

with low N-values and that a better overall match would be attained using either an intercept, such as

$q_t / (N_{60} + n^*)$ with a value $n^* = 6$ bpf, else fitting of a power function format such that $q_{t1} = \alpha \cdot [(N_{10})_{60}]^{\beta^*}$,

as suggested by Suzuki, et al. (1998). The minor improved coefficients of determination (r^2) shown in

Table 4. Undisturbed sands, triaxial friction angles, and

in-situ SPT, CPT, and DHT data. ϕ' SPT V_{s1}

Sand Triaxial type (deg) (N 1) 60 Q F R f (%) $q_{t1} \sigma'_{vo}$ (m/s)

Edo River CIDC 39.7 22 156 0.73 0.72 111 -0.08 164

Gioia Tauro CIDC, CIUC, CTX 41.5 * 30 279 0.26 0.26 174
-0.26 221

Highmont CIUC, CKoUC 41.5 5 35 0.38 0.38 44 0.11 141

Holmen CKoUC, CKoDC 33.2 1 29 0.42 0.40 28 -0.33 157

J-pit CIUC, CKoUC 32.7 3 31 0.87 0.75 20 0.19 127

Kidd CIUC, CKoUC 37.3 13 52 0.37 0.36 68 -0.02 177

Kowloon CIUC 38.1 21 55 1.84 1.80 74 0.07 NA

LL Dam CIUC, CKoUC 39.1 5 39 0.41 0.39 39 0.01 153

Massey CIUC, CKoUC 36.7 10 49 0.40 0.38 53 -0.09 168

Mildred Lake CIUC, CKoUC 39.6 18 32 0.73 0.70 74 0.02 156

Natori R. CIDC 40.9 50 226 0.30 0.30 207 -0.48 218

Tone R. CIDC 41.7 32 154 0.24 0.24 147 -0.46 203

Yodo(8) CIDC 42.4 27 194 0.95 0.94 176 -0.05 197

Yodo(10) CIDC 38.4 35 103 1.03 1.01 102 -0.19 213

Yodo(12) CIDC 39.1 31 96 0.89 0.88 106 -0.19 195

* Note: Results above for final q/p' from CTX series;
monotonic tests give 37.8 °.

NA = not available 0 50 100 150 200 250 300 0 10 20 30 40
50 60 Normalized SPT (N 1) 60 Normalized CPT q_{c1}
Yodo River Natori River Tone River Edo River Mildred
Lake Massey Kidd J-Pit LL-Dam Highmont Holmen W. Kowloon
Gioia Tauro $q_{c1} = (q_c / \sigma_{atm}) / (\sigma'_{vo} / \sigma_{atm}) 0.5$ (N 1)
) 60 = (N 1) 60 / $(\sigma'_{vo} / \sigma_{atm}) 0.5$

Figure 40. Trend between CPT and SPT data for undisturbed
sand database.

Figure 41 lend support to this notion. Alternatively, the

procedures for stress normalization of SPT and/or

CPT data may be involved in the observed trends (e.g., Boulanger & Idriss 2004; Moss, et al. 2006).

Andrus & Stokoe (2000) present a relationship between the normalized shear wave velocity,

$V_{s1} = V_s / (\sigma'_{vo} / \sigma_{atm})^{0.25}$, and the energy-corrected & stress-normalized N-value, $(N_1)_{60}$. The data from

the undisturbed sands appear to conform fairly well with this correlation, as shown by Figure 42a. Sim

ilarly, Andrus et al. (2004) developed a similar interrelationship between V_{s1} and normalized cone tip

stress (q_{t1}), as supported by the dataset from the undisturbed sands (reference Figure 42b). Thus, the

compiled in-situ data from SPT, CPT, and V_s appear internally consistent.

All of the undisturbed sands were tested under consolidated triaxial compression tests to determine

their respective effective stress friction angles (ϕ'). The measured values are reported in Table 4. This

permits an opportunity to check on the validity of selected relationships in evaluating sand strength

from available in-situ test data (Mayne 2006b). Using the aforementioned expression between ϕ' and

$(N_1)_{60}$ developed by Hatanaka & Uchida (1996), only a fair agreement is noted for the new dataset in CPT-SPT Interrelationships from Undisturbed Sands $y = 4.38x R^2 = 0.778$ $y = 3.76x + 18.4 R^2 = 0.811$ $y = 14.4x - 0.633 R^2 = 0.854$ x 50 100 150 200 250 300 x 10 20 30 40 50 60 Normalized SPT $(N_1)_{60}$ N o r m a l i z e d C P T q_{c1}

Figure 41. Regression relationships between CPT and SPT for undisturbed sand database. x 50 100 150 200 250 300 y 50 100 150 200 250 300 y 50 100 150 200 250 300 Normalized Tip Stress, $q_{t1} = q_t / (\sigma'_{vo})^{0.5}$ N o r m . S h e a r W a v e , V_{s1} (m / s) Japan Canada Norway Italy Andrus et al (2004) x 10 20 30 40 50 60 Normalized SPT $(N_1)_{60}$

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(m

/ s) Japan Canada Norway Italy Andrus et al (2004) V_{s1}
(m/s) $\approx 62.6 (q_{t1})^{0.231}$ V_{s1} (m/s) $\approx 87.8 [(N_1) 60]^{0.231}$

Figure 42. Normalized shear wave velocity vs. (a) SPT (N_1) 60 and (b) CPT q_{t1} for undisturbed sands.

comparison with their frozen sand samples from additional Japanese test sites (Figure 43). The trend

is much flatter for the new data and suggests a relative insensitivity of (N_1) 60 in accurately estimating

effective ϕ' in sands. The friction angles of loose to firm sands at low to medium N-values are significantly

underpredicted by the expression. Perhaps, the repeated dynamic nature of the SPT at low N-values is

more indicative of an “undrained” type cyclic loading. The relationship between the triaxial-measured ϕ' of undisturbed sands and normalized cone tip

resistance is presented in Figure 44. Here, the CPT proves to be an excellent predictor in representing

the drained strength of the sands. The two outliers from LL

and Highmont Dams are sands with some

unusual mineralogies beyond the normal quartzitic & feldspathic types, exhibiting higher percentages

of clay and other minerals (as noted). In fact, if the mica content were also included, LL Dam would

have 50% and Highmont would show composition of 20% of minerals other than quartz & feldspar. Another interesting comparison is the between the apparent OCRs as suggested from the SPT and

CPT data, shown in Figure 45. The agreement from these two separate approaches is quite remarkable for

all fifteen sand datapoints. Both the SPT and CPT indicate values of OCR < 1 for three sands (Holmen,

Highmont, and Mildred Lake), with values up to as high as OCR = 7 for Gioia Tauro sand. Sands

with OCRs < 1 imply unstable structure and perhaps this could be used as an indicator of liquefaction

potential. The CPT data also infers that the sands at J-pit and LL dam have OCRs ≤ 1 (although SPT

results indicate OCR > 1). This is of interest as both LL and Highmont were noted earlier to have some 20 25 30 35 40 45 50 0 10 20 30 40 50 60 Normalized N-Value, (N 1) 60 T r i a x i a l ϕ' (d e g .) Yodo River Natori River Tone River Edo River Mildred Lake Massey Kidd J-Pit LL-Dam Highmont Holmen H&U Natural Sands H&U Sand Fills W. Kowloon Gioia Tauro Uchida & Hatanaka '96 ϕ' 20° [15.4 (N 1) 60] 0.5

Figure 43. Effective friction angle of undisturbed sands vs. normalized SPT resistance in comparison with

Hatanaka & Uchida (1996) relationship. 30 35 40 45 50 0 50 100 150 200 250 300 Normalized Tip Stress, $q_{t1} / q_{t0.5}$ (v o ') 0.5 T r i a x i a l ϕ' (d e g .) Japan Canada Norway China Italy K&M'90 26 18 7 5 5 10 10 0 0 0 0 0 0 0 Total %: kaolinite, smectite, calcite, illite, and chlorite $\sigma_{vo}' \cdot \sigma_{atm}' q_{t1} \phi'$ (deg) 17.6 11.log

Figure 44. Effective triaxial friction angle of undisturbed sands vs. normalized cone tip stress.

unusual mineralogy, as compared with the other eleven sands. Also, the J-pit site was selected for the

full-scale attempt at liquefaction for the CANLEX experiments, yet did not liquefy (Robertson, et al.

2000a,b).

3.7 Stress-strain-strength response of sands

The stress-strain-strength response of sands can be handled similarly to the aforementioned approach,

using a variety of nonlinear expressions or algorithms to represent modulus reduction (G/G_{max}), or

alternatively using constitutive soil models. In the generalized modified hyperbola version by Fahey & Carter (1998), the OCRs of undisturbed sands from SPT and CPT inferences are plotted against the secant modulus reduction factors (G/G_{max}) for various sands. The data points are as follows: Yodo River, Natori River, Tone River, Edo River, Mildred Lake, Massey Kidd, J-Pit, LL-Dam, Highmont, Holmen W. Kowloon, Gioia, Tauro. The regression equation is $G/G_{max} = 1.01 \times OCR^{0.93}$ with $r^2 = 0.93$.

Figure 45. Apparent OCRs of undisturbed sands from SPT and CPT inferences.

Carter (1993), two parameters (f and g) are used to obtain the secant modulus reduction factors (Fahey,

1998) by one of the following: For the above, no changes in ν' are made during loading, however, could be implemented as needed

(e.g., Fahey & Carter, 1993). Poisson's ratio data on 6 sands presented by Lehane & Cosgove (2000)

show relatively constant ν' during loading up to about 0.1% strains, afterwards indicating increases in

ν' . The expressions of (33) have been used effectively to represent stress-strain-strength curves in simple

& torsion shear and triaxial modes for different soil types, including: clays (e.g., Elhakim & Mayne,

2003; Mayne et al. 2003), silts (Mayne et al. 1999), clean sands (Fahey 1998; Lehane & Fahey, 2002),

and silty sands (Lee, et al. 2004). For uncemented soils

that are not highly structured, adoptive values

of $f = 1$ and $g = 0.3 \pm 0.1$ have been suggested for initial estimates (Mayne 2005). The drained strength of sands depends on the particular mode of loading application. For direct simple

shear (DSS), the shear strength may be stated simply by: $\tau_{\max} = \sigma'_{vo} \cdot \tan \phi'$; while for isotropic triaxial

drained compression tests (CIDC), the peak deviator stress is obtained from: $q_{\max} = (\sigma_1 - \sigma_3)_{\max} =$

$2 \sigma'_{vo} \cdot \sin \phi' / (1 - \sin \phi')$. For torsional shear tests (TS) and anisotropically-consolidated triaxial tests

(CADC), consideration of the initial K_0 state can be made by stress path analyses. For CADC, this gives:

$q_{\max} = (\sigma_1 - \sigma_3)_{\max} = 2 K_0 \sigma'_{vo} \cdot [1/(1/\sin \phi' - 1)]$. The use of the modified hyperbola for representing stress-strain-strength curves of sand is illustrated

in Figure 46 for Natori River sand reported by Mimura (2003). Here, $f = 1$ and an exponent value $g = 0.3$

to 0.4 gives an overall good fitting. While the aforementioned fitting technique was shown to be simple and reasonable for a given sand,

it cannot effectively address complex stress paths and behavior, because of its empirical basis. The use

of a constitutive soil model would therefore be beneficial since it may be capable of handling additional

facets of soil behavior, including: drained and undrained loading, variable stress paths, volumetric strain

predictions & dilatancy, cyclic response, and post-peak softening. Summaries of some available models

are given by Duncan (1994) and Lade (2005). Some constitutive models of interest include the rather versatile NorSand (Jefferies, 1993; Jefferies &

Shuttle, 2005) which requires only 8 soil parameters and has been calibrated with several sands and clays.

Also, the more sophisticated MIT-S-1 model (Pestana &

Whittle, 1999) can produce a variety of stress

paths, modes, and varied responses for both sands and clays. Consequently, future research should be

directed towards the calibration of laboratory and in-situ tests within the framework of constitutive soil models. Figure 46 shows the triaxial stress-strain curve for Natori River Sand (0.50 to 0.80) and the modified hyperbola fitting (data from Mimura 2003).

Strain (%)	Deviatoric stress, q (kPa)
0	0
50	100
100	150
150	200
200	250
250	300
300	350
350	400
400	450
450	500
500	550
550	600
600	650
650	700
700	750
750	800
800	850
850	900
900	950
950	1000
1000	1050
1050	1100
1100	1150
1150	1200
1200	1250
1250	1300
1300	1350
1350	1400
1400	1450
1450	1500
1500	1550
1550	1600
1600	1650
1650	1700
1700	1750
1750	1800
1800	1850
1850	1900
1900	1950
1950	2000
2000	2050
2050	2100
2100	2150
2150	2200
2200	2250
2250	2300
2300	2350
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2850	2900
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3000	3050
3050	3100
3100	3150
3150	3200
3200	3250
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3950	4000
4000	4050
4050	4100
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4250	4300
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4950	5000
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9150	9200
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9250	9300
9300	9350
9350	9400
9400	9450
9450	9500
9500	9550
9550	9600
9600	9650
9650	9700
9700	9750
9750	9800
9800	9850
9850	9900
9900	9950
9950	10000

Figure 46. Triaxial stress-strain curve from undisturbed sand represented by modified hyperbola fitting (data from Mimura 2003).

models as these will allow versatility. The initiation of this database on undisturbed sands will help begin

the important calibration of parameters as needed for these constitutive soil models.

4 IN-SITU TESTING OF INTERMEDIATE AND NONTEXTBOOK GEOMATERIALS

4.1 Other soil types

In nature, there are enumerable types of soils with varying components, packing arrangements, and

origins. Yet, in the “ivory tower” of academe, it is usual to group soil behavior into two broad categories

(as has been covered herein): sands (drained behavior) and clays (undrained behavior). Thus, anything

outside of the “hourglass sands” and “vanilla clay” categories could be termed nontextbook geomaterials.

Within the CSSM context, all soils (clays, silts, sands, gravels) can exhibit drained to semi-drained

to partially-undrained to fully-undrained conditions, given the particular circumstances. Mixed soils

with varying percent fines, weathering, high structure, fabric, unusual mineralogical components, and

particles have been encountered in geotechnical practice

and do not fit within the empirical domain of

existing correlations and procedures of analysis. As such, a consensus version of CSSM framework for

unsaturated structured soils based in micromechanics would be desired to account for many additional

facets of soil behavior (e.g. Cho & Santamarina, 2001; Leroueil & Hight, 2003) and is really beyond the

scope & capabilities of this paper.

A number of recent papers have addressed the concerns and significance of in-situ test interpretation

in nontextbook geomaterials. Such geomaterials are widespread and diverse, including: calcareous soils,

carbonate sands, partially-saturated soils, diatomaceous clays, weathered residuum, lateritic soils, loess,

mine tailings, and gaseous sediments. For these issues, the reader is directed towards the efforts reported

Schnaid (2005), as well as the many individual papers found in the 2003 and 2006 Singapore Workshop

proceedings.

4.2 Global correlations

Of interest to the post-processing of in-situ data is the development of generalized correlations, as these

may find use in the assessment of geotechnical parameters for all types of "well-behaved" soils. Soil

behavioral charts have been produced that utilize in-situ readings and indirectly assess soil type. For

instance, the DMT readings p_0 and p_1 give the material index I_D that determines type of soil. Two readings

from CPT (q_t and f_s) can be used to give approximate soil types in low sensitivity and noncemented

geomaterials (e.g. Schmertmann 1978), else in CPTU, two readings could focus on q_t and B_q (Senneset 10 12 14 16 18 20 22 24 26 28 10 100 1000 Normalized Shear Wave

Velocity, V_{s1} (m/s) Saturated unit weight, γ_T (kN/m³) Peat Intact Clays Fissured Clays Silts Sands Gravels Weathered Rx Undisturbed Sands Regression
 Regression: $\gamma_{SAT} = 4.17 \cdot \ln(V_{s1}) - 4.03$ $n = 731$ $r^2 = 0.775$

Figure 47. Saturated unit weight evaluation from stress-normalized V_{s1} in non-cemented soils.

et al. 1988). In fact, all three readings from CPTU (q_t , f_s , and u) can be used for soil behavioral

identification (e.g., Robertson 1990). These comparison of intra-measurements can be utilized to find “unusual” or anomalous soils that

would classify as problematic geomaterials. For instance, Fahey (1998) and Schnaid et al. (2004) plot

the ratio G_0/q_t which decreases with q_{t1} for unaged quartz and siliceous sandy soils. This baseline is

then used to distinguish cemented or structured soils, as these tend to fall above this curve. Thus a

cross-comparison of multiple readings taken during the same sounding helps to identify the soil type.

With the seismic piezocone, four parameters are obtained (Q , F , B_q , $G_0./q_t$) to allow a better means to

notice or define prospective nontextbook soils. From a global database on soils ($n = 731$), the saturated unit weight (γ_{sat}) of “well-behaved” soils

can be guestimated from the stress-normalized shear wave velocity (V_{s1}) within about ± 1 kN/m³, as

suggested by Figure 46. The figure also includes the 14 undisturbed sands from Tables 2 to 4. Unit weights

from 10 to 26 kN/m³ are associated with normalized shear wave velocities from 30 to 800 m/s. As V_{s1} is

dependent upon the current effective overburden stress, the procedure should be used in a depth-stepwise

fashion, starting from the ground surface and proceeding downward. It can be expected that cemented

soils and carbonate sands would lie somewhat to the right of the mean relationships shown, as the bonding

would promote a more open porous structure (and low unit weight), yet a fast velocity in the matrix. For dry soils above the water table and no capillarity effects, a similar relationship can be derived

from lab resonant column test data. Figure 47 shows the reprocessed RCT data from four series of

reconstituted quartz sands tests (Richart et al. 1970) in terms of dry unit weight vs. V_{s1} . Dry unit weights

from 14 to 20 kN/m³ are associated with normalized V_{s1} from 220 to 280 m/s. The real difficulty with this indirect approach comes with partially-saturated soils in the field. Here,

use of measured shear wave velocities in the vadoze zone above the groundwater table is complicated

because of capillarity effects. As the soil is desaturated, the shear wave velocity increases by two-fold to

five-fold, the extent depending upon the grain size distribution of the soil (Cho & Santamarina, 2001).

Thus, a measure of the degree of saturation may be necessary. Towards this purpose, Jamiolkowski

(2001) has suggested that in-situ compression wave measurements (V_p) can aid in evaluating the degree

of saturation. With many well-documented geotechnical experimental test sites now established (such as those cases

reported in the 2003 and 2006 Singapore Workshops), it is now feasible to seek comparisons across diverse

soil types and formations, looking for more global (and presumably, more reliable) correlations that can

serve as baselines in cross-checking site-specific data and generalized interpretation of soil engineering

parameters. The author has made some initial steps towards this purpose using compiled SCPTU data

from a variety of clays, silts, and sands, including many reported in the first Singapore proceedings (Tan

et al. 2003). In addition to intact clays, the selection considers fissured clays, organic clays, and silts. Dry Rounded Quartz Sands (Richart, Hall, & Woods 1970) 12 13 14 15 16 17 18 19 20 21 22 100 150 200 250 300 350 400 Normalized V_{s1} (m/s) Dry Unit Weight, γ_d (kN / m³) Sand A - Gap Graded Sand B - Well Graded Sand C - No. 80 to 140 Sand D - No. 20 - 30 Trendline $0.25 \gamma_d / (V_{s1}^2)$ (m/s) V_{s1} (m/s) = $\sigma_{atm} \sigma'_{vo} \gamma T = 2 + 0.06 V_{s1} r^2$ = 0.719 n = 104

Figure 48. Trend of dry unit weight of sands in terms of stress-normalized shear wave velocity. Amherst Bangkok AIT Bothkennar Brent Cross Cowden UK Drammen Fucino Holmen Sand Lilla Mellosa Madingley Montalto Onsoy Opelika NGES Pentre Piedmont Silt Pisa Ska Edeby Evanston NC Ticino OC Ticino NC Toyoura Sand OC Toyoura Sand Canada Sands Japanese Sands 0.1 1 10

100 1 10 100 1000 Sleeve Friction, f_s (kPa)

C o

n e

T

i p

S

t r e

s s

, q

t

(M

P a

) Intact CLAYS SANDS Fissured CLAYS SILTS Organic plastic CLAYS

Figure 49. Combinative SCPTU database with q_t vs. f_s summary for varied clays, silts, and sands.

A collection of cross-comparisons from the CPTs from these sites is shown in terms of cone tip resistance

vs. friction resistance in Figure 49. The undisturbed sands from Tables 2 to 4 are also included.

For foundation settlement analyses, a representative constrained modulus of the supporting soil

medium is usually sought. The one-dimensional consolidation test (or oedometer) provides the con

strained modulus ($D' = \sigma' / \epsilon$), although in some cases, the modulus values can be backcalculated

from field performance data from embankment loadings or footings. In practice, it has been usual to

further correlate the modulus D' to a penetration resistance, as in-situ penetration tests are the most

common type employed in routine site investigations. From the global plotting of diverse geomaterials,

Figure 50a shows that a relationship for “well-behaved” soils might take the form (Schmertmann 1978):

with a representative value of $\alpha' C \approx 5$ for soft to firm “vanilla clays” and NC “hourglass sands”. However,

for organic plastic clays of Sweden, a considerably lower $\alpha' C \approx 1$ to 2 may be appropriate. For cemented

Fucino clay, a value $\alpha' C \approx 10$ to 20 may be assigned. 0.1
 1 10 100 1000 1 10 100 1000 Small-Strain Shear Modulus, G_0
 (MPa) C o n s t r a i n e d M o d u l u s , D' (M P a)
 0.1 1 10 100 1000 0.1 101 100 Net Cone Tip Stress, $q_t - \sigma_{vo}$
 (MPa) C o n s t r a i n e d M o d u l u s , D' (M P a) SANDS Intact CLAYS SANDS Intact CLAYS Organic plastic
 CLAYS Organic plastic CLAYS Cemented cemented SILTS SILTS
 Ratio $D'/G_0 = 2$ 0.7 0.2 0.1 0.02 fissured clays $D' = 5 (q_t - \sigma_{vo})$

Figure 50. Combinative soil SCPTU database relationships between oedometric modulus (D') and (a) net cone tip

resistance and (b) small-strain shear modulus. 0 100 200
 300 400 1 10 100 1000 Sleeve Friction, f S-CPT (kPa) S h e
 a r W a v e V e l o c i t y , v_s (m / s) CLAYS SANDS

Regression: $n = 161$ $V_s = 51.6 \cdot \ln(f_s) + 18.5$ with V_s (m/s) and f_s (kPa) $r^2 = 0.823$ Organic plastic CLAYS 10 100 1000 0.1 101 100 Net Cone Tip Stress, $q_t - \sigma_{vo}$ (MPa) Small Strain Shear Modulus, G_0 (MPa) CLAYS SANDS Clays: $G_0 = 50 (q_t - \sigma_{vo})$ (Tanaka and Tanaka, 1999) σ_{atm} $q_t - \sigma_{vo}$ Sands: $G_0 = 50 \cdot \sigma_{atm}$ σ_{vo} cemented SILTS Organic plastic CLAYS 0.6

Figure 51. Observed SCPTU trends for non-calcareous soils: (a) shear wave vs. sleeve friction; and (b) small-strain

stiffness and net cone tip resistance. An alternate correlation can be sought between D' and small-strain shear modulus (G_0), as presented

in Figure 50b. In this case, a similar adopted format could be:

with assigned values of $\alpha' G$ ranging from 0.02 for the organic plastic clays up to 2 for overconsolidated

quartz sands. Additional studies with multiple regression, artificial neural networks, and numerical

modeling may help guide the development of more universally-applied global relationships. During the examination of various possible relationships in the above soil database, the results of

two interesting trends became apparent (Figure 51). In Figure 51a, the measured V_s appeared related to

the sleeve friction for uncemented soils. Figure 51b showed that the proposed relationship by Tanaka &

Tanaka (1999) could be modified for soil type according to:

where $m^* = 0.6$ for clean quartz sands, 0.8 for silts, and 1.0 for intact clays of low to medium sensitivity.

Note the remarkable resemblance to the aforementioned (30).

Figure 52. Concept for an integrated geotechnical approach to ground investigations.

5 CONCLUSIONS

The interpretation of in-situ tests in soils is currently a mix & match of theoretical, analytical, and empir

ical relationships that have developed separately and independently based on single-focused needs and

research studies. Yet now, a sufficiently high number of well-documented clay and sand sites currently

exists to allow a comprehensive calibration of a full suite of in-situ tests within a consistent framework.

The results can be validated using high-quality laboratory test data and/or parameters backfigured from

full-scale load tests. What is needed is a unified framework based on numerical modeling of all major test

types using a rigorous constitutive soil model (likely within the context of critical state soil mechanics).

This should be an integrated approach using geophysics, drilling & sampling, laboratory and in-situ

testing (Fig. 52).

In the interim, this paper shows that stress history (i.e., OCR and OCD) can play a significant role

in unifying the approach to in-situ test interpretations in soils. As a suggested first step, the utilization

of a simplified cavity expansion - CSSM approach to fitting CPT, CPT_u, and DMT data to clay sites is

presented for six sites that range from soft NC to stiff OC states. For clean sands, an empirical approach

using SPT, CPT, and DHT V_s data is applied to several well-documented sands and a special undisturbed

(frozen) sand database is presented to cross-check relationships developed from large scale calibra

tion chamber test series. Procedures for nontextbook geomaterials are also in dire need of a common

methodology that can address clays and sands, as well as intermediate soil types and unusual to difficult

soils. Additional global relationships for "well-behaved" and "normal" soils are sought in order to iden

tify problematic and “ill-behaved” geomaterials. In-situ tests which obtain multiple-measurements (i.e.,

SCPT and SDMT) offer the most promise as intra-comparative cross-checks amongst the readings can

be used to help discern anomalous soil behavior, as well as provide a suite of engineering parameters

for “normal” soils, including stress-strain-strength-flow characteristics. Consequently, it is quite evident

that considerable work remains in the development of a rational and validated approach to geotechnical

site characterization by in-situ tests.

ACKNOWLEDGMENTS

The author appreciates the financial support provided currently by the Georgia Dept. of Transportation

(GDOT), as well as prior support given by the National Science Foundation (NSF) and Fugro Engineers

BV. Any conclusions or findings reported herein do not necessarily reflect the procedures or guidelines

of these groups.

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Characterisation and Engineering Properties of Natural Soils - Tan, Phoon, Hight & Leroueil (eds) © 2007 Taylor & Francis Group, London, ISBN 978-0-415-42691-6

Soil variability analysis for geotechnical practice

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ABSTRACT: The heterogeneity of natural soils is well known in geotechnical practice. However, the

importance of quantifying the resulting variability in geotechnical characterisation and design parameters

is not adequately recognised. Variability should not be approached suspiciously. Rather, it should be

accepted as a positive contribution to geotechnical design as its consistent modelling and utilisation

lead, with limited additional computations and conceptual effort on the part of the engineer, to more

rational and economic design. The paper presents a structured - though necessarily partial - review of

approaches and methodologies for the quantification of soil variability, as well as selected examples of

its utilisation in reliability-based geotechnical design.

1 INTRODUCTION

Soils are naturally variable because of the way they are formed and the continuous processes of the

environment that alter them. After deposition or initial formation, they are modified continuously by

external stresses, weathering, chemical reactions, introduction of new substances and, in some cases,

human intervention (e.g., soil improvement, excavation, filling).

In a geotechnical perspective, three levels of soil heterogeneity could be defined. Stratigraphic het

erogeneity is the result of large-scale geologic and geomorphological processes. This heterogeneity is

usually addressed at site-scale; stratigraphies may be extremely complex and heterogeneous. This level of

heterogeneity is commonly addressed by the geotechnical engineer in the context of site characterisation,

in which the geotechnical engineer expects the existence of soil units which are strongly heterogeneous

from a compositional or mechanical point of view.

Lithological heterogeneity can be manifested, for instance, in the form of thin soft/stiff layers embed

ded in a stiffer/softer media or the inclusion of pockets of different lithology within a relatively uniform

soil mass.

Inherent soil variability is the variation of properties from one spatial location to another inside a

soil mass which could be regarded as being significantly homogeneous for geotechnical purposes. At

this level, it is necessary to assign quantitative values to parameters of interest; such values should be

representative of the parameters in the soil unit.

The conceptual relevance of recognising soil heterogeneity does not decrease on a smaller scale. The

roles of the three types of heterogeneity in the engineering process are significantly different. Large

scale stratigraphic heterogeneity is important in the characterisation phase and for preliminary design

decisions; medium-scale lithological heterogeneity may affect the choice of the engineering model to

obtain a design value for a parameter of interest; a proper analysis of inherent variability can become a

powerful tool in the hands of the engineer to achieve more rational and economic design.

The term "inherent variability" used in this paper only applies to scales measurable by com

mon geotechnical testing. There is currently a knowledge gap between our physical understanding

of microstructure and how statistical variations at micro-scale impact engineering-scale systems. For

instance, the modulus degradation curve provides us with a glimpse of the underlying complexities

in natural soils. Comparable variations in strains do not translate to comparable variations in modulus

because of nonlinear behaviour. Hence, variations in modulus deduced from large-strain in-situ tests

cannot be extrapolated to variations at smaller scales. Nevertheless, it is well known that ground settle

ment behind deep excavations depend on small-strain stiffness and variations in ground settlement would

thus depend on variations in small-strain stiffness. These variations potentially can be characterized by

shear wave velocity and comparable small-strain non-intrusive measurements (e.g., seismic CPT) that

are gradually adopted in practice. The engineer processes available data to obtain parameters which are useful for characterisation and

design. Far from being a merely technical issue, the choice of the approach by which a geotechnical

engineer creates a model of physical reality as he is given to see: from data which are never completely

precise and accurate, reflects a philosophical choice which is, nonetheless, heavily dependent on - and

constrained by - regulatory, economic and technological factors. Vick (2002) addressed the complex

universe of decision-making in geotechnical reasoning and its many nuances. The choice of representative parameters, whichever the geotechnical approach, recognises the inherent

variability of soils. However, the level of explicitness with which variability is included in the assignment

procedure, depends upon the selected approach. In deterministic approaches, inherent variability does

not appear as explicitly as in uncertainty-based approaches. In principle, no category of approaches is preferable over another. The quality of geotechnical char

acterisation or design is not conceptually bound to the level of explicitness of soil variability modelling.

Deterministic methods lie at the basis of virtually every technological science, and geotechnical engin

earing is no exception. However, collective experience

(both from practice and research) suggests that it may be time for a shift to an uncertainty-based perspective which may be, on the whole more convenient in terms of safety, performance and economy. The main advantage of uncertainty-based methods over deterministic methods in terms of safety lies in the fact that, due to the explicit inclusion of uncertainty in inputs and the explicit declaration of the level of uncertainty in the outputs, the former are able to provide more complete and realistic information regarding the level of safety of design: soils are variable, whether such variability is recognised in design or not! Addressing uncertainty does not increase the level of safety, but allows a more rational design as the engineer can consciously calibrate his decisions on a desired or required performance level of a structure. Being able to select and communicate the level of performance and reduce undesired conservatism, in turn, is beneficial in the economic sense. Hence, geotechnical practitioners should envisage soil variability as a positive factor for design. Perhaps the main trade-off for the positive aspects of uncertainty-based approaches, which has hindered its dissemination among geotechnical practitioners, is the necessity to rely on somewhat more complex estimation approaches involving. It is necessary to refer to mathematical techniques which address uncertainty. While ever-increasing computational capabilities and new dedicated software gradually remove some of the barriers encountered by research and practitioners in the past (e.g. the computational expense of implementing Monte Carlo simulation), variability analysis requires at least some degree of comprehension on the side of the engineer

who performs it or even assesses its results.

Engineering decision and judgment should never be completely replaced by automated computational

procedures, however powerful these may be.

1.1 Variability vs. uncertainty

In the engineering literature - and geotechnical engineering is no exception - the terms variability and

uncertainty are often employed interchangeably. While it is true that the two terms refer to concepts which

are significantly related, and that the frequently occurring terminological superposition is unlikely to

result, in practice, in unfavourable consequences, it is opined here that a clarification would positively

contribute to a progressive, virtuous increase in the terminological rigour in the geotechnical literature. In the technical sense, variability can be defined as an observable manifestation of heterogeneity of

one or more physical parameters and/or processes. In principle, a variable property could be described,

for instance, if a sufficient number of measurements were available and if the quality of the measurements

themselves was sufficient to ensure confident evaluation of the observations. Hence, the observation

of variability implicitly provides a more or less detailed assessment of the level of knowledge on a

phenomenon of interest and of the capability to measure and model the phenomenon itself. Uncertainty reflects the decision (or necessity) to recognise and address the observed variability in

one or more soil properties of interest. In many cases it is not possible (given the inherent nature of a

physical process and the available level of knowledge and/or technology related to the process itself) to

model the variability in a completely satisfactory way. On

the other hand, anticipating a fundamental

concept of soil variability modelling, it may be deemed advantageous to assume that a phenomenon

is indeterminate (uncertain) to a certain extent in cases where a detailed description of variability is

expected to be uneconomic or redundant.

1.2 Applications of soil variability modelling

The main practical applications of soil variability modelling are: (a) geostatistics, focusing on inter

polation of available data values to estimate other unavailable data values at the same location; and

(b) geotechnical engineering, that focuses on characterisation for reliability/risk assessment. While the

theoretical aspects underlying such applications are not the topic of the paper, it is deemed useful to

provide a concise description of each.

1.2.1 Geostatistics

A well known problem in geotechnical site characterisation is the limited amount of available data. It is

frequently necessary to estimate soil properties at specific locations where they have not been observed.

To do so, it is necessary to interpolate the available data while accounting for the spatial correlation

of soil properties of interest. Geostatistics makes use of a specific modelling approach to such spatial

interpolation known as geostatistical kriging.

Kriging is essentially a weighted, moving average interpolation procedure that minimises the estimated

variance of the interpolated value with the weighted average of its neighbours. The input information

required for kriging includes: available data values and their spatial measurement locations; information

regarding the spatial correlation structure of the soil property of interest; and the spatial locations of

target points, where estimates of the soil property are desired. The weighting factors and the variance are

computed using the information regarding the spatial correlation structure of the available data. Since

spatial correlation is related to distance, the weights depend on the spatial location of the points of

interest for estimation.

The works of G. Matheron, D.G. Krige, and F.P. Agterberg in founding this area are notable. It should

be acknowledged that parallel developments also took place in meteorology (L.S. Gandin) and forestry

(B. Matérn). Finally, geostatistics is fundamentally based on the theory of random processes/fields

developed by mathematicians A.Y. Khinchin, A.N. Kolmogorov, P. Lévy, N. Wiener, and A.M. Yaglom,

among others. Many forms of kriging have been developed since; the reader is referred to, for instance,

Journel & Huijbregts (1978), Davis (1986) and Carr (1995).

Nadim (1988) and Lacasse & Nadim (1996) provided applications of geostatistical kriging. Figure 1a

(Lacasse & Nadim 1996) shows the locations of cone penetration test soundings in the neighbourhood

of a circular-shaped shallow foundation which was to be designed; Figure 1b reports the superimposed

profiles of cone resistance in the soundings. It was of interest to estimate the values of cone resistance in

other spatial locations under the design location of the foundation itself. Figure 2 presents the contours of

cone penetration resistance at each metre as obtained by kriging. The 3D graphic representation provides

improved insight into the possible spatial variation of cone resistance and the most likely values beneath

the foundation. The results of the analysis enabled designers to determine more reliably the position of

a clay layer and to use higher shear strength in design.

1.2.2 Reliability-based geotechnical design

The practical end point of characterising uncertainties in the design input parameters (geotechnical,

geo-hydrological, geometrical, and possibly thermal) is to evaluate their impact on the performance of

a design. Reliability analysis focuses on the most important aspect of performance, namely the prob

ability of failure (where “failure” is a generic term for non-performance). This probability of failure

clearly depends on both parametric and model uncertainties. The latter are not covered in this paper.

The probability of failure is a more consistent and complete measure of safety because it is invariant

to all mechanically equivalent definitions of safety and it incorporates additional uncertainty informa

tion. There is a prevalent misconception that reliability-based design is “new”. All experienced engineers

would conduct parametric studies when confidence in the choice of deterministic input values is lacking.

Reliability analysis merely allows the engineer to carry out a much broader range of parametric studies

without actually performing thousands of design checks with different inputs one at a time. This sounds

suspiciously like a “free lunch”, but exceedingly clever probabilistic techniques do exist to compute the

probability of failure efficiently. The chief drawback is that these techniques are difficult to understand

for the non-specialist, even though they may not necessarily be difficult to implement computationally.

There is no consensus within the geotechnical community if a more consistent and complete measure

of safety is worth the additional efforts or if the significantly simpler but inconsistent global factor of

safety should be dropped. Regulatory pressure appears to be pushing towards reliability-based design for (a) (b)

Figure 1. (a) Locations of cone penetration test soundings; (b) Superimposed profiles of cone resistance (Lacasse &

Nadim 1996).

Figure 2. Contours of cone penetration resistance at each metre as obtained by geostatistical kriging (Lacasse &

Nadim 1996).

Table 1. Factors determining the character of geomaterials (Hight & Leroueil 2003).

Formative history

Sedimentary

Depositional environment (alluvial, marine, Lacustrine, glacial, estuarine)

Post-depositional processes (cementing, bioturbation, leaching, desiccation, ageing, tectonics, lithification, chemical alteration, weathering)

Residual

Form and intensity of weathering

Parent rock

Age

Composition

Complete grading

Grain/aggregate size, shape, texture and strength

Silt fraction - shape - plasticity

Clay fraction - mineralogy - form (clay minerals, rock flour, biogenic debris)

Stability

Organic content and form

Pore water chemistry (salinity, sulphates, pH, etc.)

Fabric

Macrofabric

Interbedding, laminations, discontinuities (faults, joints, fissures, open cracks)

Microfabric

Orientation, density variations, bioturbation features, pore size distributions, local void ratio

Microstructure

Cementation - form, distribution, strength

Ageing effects (including creep)

Recent stress/strain history

Unloading/reloading

Groundwater level fluctuations

Wave loading

Ground movements

Sampling

State

Water content, void ratio, degree of saturation

Density, relative density

In-situ stresses (including pore pressure) and location relative to limit state curve

Current strain rate and time effects

Drainage conditions

Drained, partially drained, undrained (permeability, drainage path lengths, rate of stress [or strain]

change, pore pressure gradients)

Stress/strain path imposed by construction and subsequent loading

Rate of stress or strain change

Disturbance/destructuring

Temperature

non-technical reasons. The literature on reliability analysis and reliability-based design is voluminous.

The reader is referred to Phoon et al. (2003a, 2003b) for a summary and more detailed reference listings.

1.3 Factors inducing soil variability

The spatial variation of a soil property is believed to be more related to formation processes rather

than to the chemical and mechanical details of the soil particles themselves. This has been asserted

both for natural soils and for man-made soils (e.g. Fenton 1999a). Larsson et al. (2005) specifically

addressed variability in man-made soils. Table 1 (Hight & Leroueil 2003) provides a list of the main

factors contributing to the definition of the character of geomaterials.

Spatial variability may also be influenced by artificial disturbances such as construction or induced

which are relevant to the definition of the spatial correlation structure of a soil property.

The present paper focuses on the quantitative modelling of inherent soil variability; hence, the physical

phenomena contributing to variability will be addressed, when pertinent, in the light of their effect on

quantitative estimation. The interested reader is referred to Hight & Leroueil (2003) and Locat et al.

(2003) for further insights into the physical causes of variability.

1.4 Geotechnical uncertainty

The total uncertainty in a measured or design geotechnical parameter may be addressed quantitatively

by uncertainty models. Several models have been proposed in the literature (e.g. Phoon & Kulhawy

1999b). Uncertainty models identify inherent soil variability, measurement error, statistical estimation

error and transformation error as the primary sources of uncertainty. Inherent variability gives rise to

aleatory uncertainty, while measurement error, statistical estimation error and model error contribute to

epistemic uncertainty (e.g. Lacasse & Nadim 1996; Phoon & Kulhawy 1999a). The present paper focuses

on inherent soil variability or, in other words, on the quantification and effects of aleatory uncertainty

on geotechnical characterisation and design. It is deemed useful to describe the components of epistemic uncertainty in extreme synthesis. Measurement

uncertainty is due to equipment, procedural-operator, and random testing effects. Equipment

effects result from inaccuracies in the measuring devices and variations in equipment geometries and

systems employed for routine testing. Procedural/operator effects originate from limitations in test stand

ards and how they are followed. Random measurement error refers to the remaining scatter in the test

results that is not assignable to specific testing parameters and is not caused by inherent soil variability.

It is usually possible to identify a measurement bias (i.e. a consistent overestimation or underestimation

of the real value of the object parameter) and a model dispersion. Orchant et al. (1988) discussed

the factors contributing to measurement uncertainty for geotechnical laboratory and in-situ testing.

Kulhawy & Trautmann (1996) and Phoon & Kulhawy (1999a) provided tabulated values of measurement

uncertainties for laboratory and in-situ tests. Transformation uncertainty results from the approximations which are inevitably present in empirical,

semi-empirical or theoretical models commonly used in geotechnical engineering to relate measured

quantities to design parameters. This transformation or model error may include a model bias and a

model dispersion. Such measures of uncertainty are of great importance in uncertainty-based design

approaches such as reliability-based design. Model statistics can only be evaluated by: (a) realistically

large-scale prototype tests; (b) a sufficiently large and representative database; and (c) reasonably high

quality testing in which extraneous uncertainties are well controlled. Generally, available testing data

are insufficient in quantity and quality to perform robust statistical assessment of model error in most

geotechnical calculation models. The reader is referred to Ang & Tang (1975), Baecher & Christian

(2003) and Phoon & Kulhawy (2005) for a more detailed insight into model uncertainty. Data sets are always finite in size. As will be discussed in §2, statistical inferences from any finite

set of data are affected by statistical estimation uncertainty. The magnitude of this uncertainty may be

relevant in the case of small data sets. In quantitative

uncertainty models, the various components of uncertainty are assumed to be uncor-

related. However, this is only approximately true. The quantification of spatial variability requires data

from measurements of soil properties of interest. Different geotechnical measurement methods, whether

performed in the laboratory or in-situ, will generally induce different failure modes in a volume of

soil. This usually results in different values of the same measured property. Measured data are conse-

quently related to test-specific failure modes. The type and magnitude of spatial variability cannot thus

be regarded as being inherent properties of a soil volume, but are related to the type of measurement

technique. The hypothesis of uncorrelated uncertainty sources, though approximate, is especially important as it

justifies separate treatment of inherent soil variability and the remaining components of total variability. In terms of uncertainty, the aleatory uncertainty related to the inherent variability of soils is signifi-

cantly different from the epistemic uncertainty: while the latter can be reduced by improving the quality

of data and models (i.e. decreasing measurement and transformation uncertainty, respectively) and col-

lecting more data (i.e. reducing statistical estimation error), aleatory uncertainty cannot be reduced, and

can even increase as a consequence of an increase in the amount of data. Results of past research (e.g. Orchant et al. 1988) indicate that the determination of aleatory uncertainty

is significantly more reliable than the direct quantification of epistemic uncertainty.

1.5 Objectives and structure of paper

Inherent soil variability is a vast and complex topic. Even the investigation of a facet of variability,

namely its modelling in an uncertainty-based perspective,
is excessively ambitious if comprehensiveness

is sought. Far from attempting to provide a thorough review
of the available literature, this paper

aims to:

- Define spatial variability in the context of other types
of variability encountered in the geotechnical

discipline;

- Communicate the importance and convenience of
investigating spatial variability;
- Provide an applicative insight into statistical and
probabilistic modelling of geotechnical data;
- Illustrate a number of commonly used techniques for the
quantitative description of spatial variation

of soil properties;

- Report techniques which allow modelling of important
physical phenomena in quantifying the effects

of inherent variability on engineering design;

- Explain the existence of different levels of analysis of
soil variability, identifying advantages and

disadvantages in the context of design;

- Provide example applications of reliability-based
techniques making use of the results of variability

analyses;

- Provide updated tables of literature values of parameters
of variability at the different levels of analysis

while highlighting the limitations in the domain of
applicability of literature data.

The rationale is to define a “best-practice” framework for
the quantification and the communication of

inherent soil variability parameters. The specific

constitutive phases of a best-practice procedure are

not static in time (research is ever ongoing) and are not univocally defined, as the choice of the most

appropriate technique to address a specific problem is bound to the characteristics of available data.

Quantitative estimation of variability must forcedly rely on techniques which address uncertainty.

Here, methods from statistics and probability theory as well as time series analysis are referred to. As

this paper aims to stimulate the use of such methodologies by practicing engineers, the attempt is made

to reduce the mathematical formalism as much as possible, ensuring a format suitable for application

purposes.

The definition of common and consistent terminology and mathematical notation is a difficult task,

given the formal and substantial heterogeneity in the many disciplines which contribute to soil variability

investigations. Nonetheless, no effort was spared to achieve this goal and improve the flow and readability

of the paper.

Data tables containing literature values of relevant output parameters of inherent variability are pro

vided. However, such data should only be viewed as an example, and not used for reference of application

purposes, unless it is verified that the target site in which they are being applied is sufficiently similar to

the source site from which it was obtained. As important information regarding the in-situ characteristics

of the source site and relevant details of the techniques are seldom specified even in research efforts, it

is generally incorrect to use single, specific literature numerical values uncritically. It is more useful to

view such data as a plausible range of values.

Investigation on soil variability can be performed at various levels of complexity, depending on the

scope of the analysis, the quantity and quality of available data and the computing resources available.

At present, a “best-practice” soil variability investigation should address, for a parameter of interest,

at least: (a) classical descriptive and inferential statistical analysis (e.g. estimation of mean, coefficient

of variation and probability distribution); (b) spatial correlation structure describing the variation of the

soil property from one point to another in space; (c) identification of the magnitude of spatial continuity,

beyond which no or small correlation between soil data exists; and (d) spatial averaging and variance

reduction, which help assess the reduction in the variance upon averaging over a volume of interest.

Phases (a)-(d) are listed in increasing order of complexity, and can be addressed using well established

mathematical techniques from statistics and time series analysis. The phases are sequential in the sense

that higher-level analyses require the results of lower-level analyses.

The trade-off for the simplicity of lower-level analyses lies in the limited generality of results as well

as in the incomplete modelling of the behaviour of geotechnical systems. Reliability-based design can

be achieved for different levels of complexity; however, higher levels of complexity allow consideration

of additional effects and parameters which improve the quality of the reliability estimates.

The structure of the paper essentially reflects the ideal progression in complexity of a best-practice

soil variability analysis. Section 2 briefly reviews descriptive and inferential statistical techniques for

probabilistic modelling of soil properties in the second-moment sense. Section 3 offers a review of the

most commonly adopted methods for the modelling of spatial variability of soil properties and for the

identification of the spatial correlation structure. Section 4 addresses the parameters which are able

to describe a spatially variable set of data concisely in the context of random field theory, as well as

introducing the important concepts of spatial averaging and variance reduction. Section 5 and Section 6

address in detail two fundamental aspects of soil variability analysis: respectively, the identification of

physically homogeneous soil units and the verification of statistical independence of data sets. In recent years, integrated approaches making use of Monte Carlo simulation, finite element analysis

and the results of high-level soil variability investigations have been proposed. These approaches allow

enhanced modelling of the real behaviour of geotechnical systems, in which the spatial heterogeneity

of soil properties invariably plays an important role. While these studies are prevalently confined to a

research perspective at present, they may provide very useful support to code-writers and practitioners

in the future. An overview of applications of such techniques is provided in Section 7. Section 8 offers some closing remarks. Sections 2, 3, 4 and 7 are conceptually sequential, as they address soil variability modelling with

increasing complexity, consistently with the aforementioned levels of analysis. A number of fundamental

assumptions regarding the physical and statistical homogeneity of soils are common to all levels; these

are addressed in Section 5 and 6, respectively. The contents of Sections 2, 3 and 4 can be used both for geostatistical analyses and for reliability-based

design. The practical implementation of the soil variability techniques presented herein is not the central

topic of the present paper. A comparative review of reliability-based slope stability analyses from the

geotechnical literature are provided at the end of Sections 2, 4 and 7. Section 7 also reports literature

examples for other geotechnical system typologies such as soil-foundation systems, water-retaining earth

structures, underground pillars and liquefaction-prone soil masses.

2 STATISTICAL AND PROBABILISTIC MODELLING OF SOIL VARIABILITY

Among available techniques for the investigation of uncertainty, the combined use of probability and

statistics is the most frequently encountered in the geotechnical engineering literature. The reasons for

the widespread reference to such approach are likely to be manifold. Probability and statistics rely on a

very consistent bulk of literature; moreover, many concepts are intuitive to some degree.

2.1 Probability and statistics for uncertainty analysis

Probability and statistics form two distinct branches of mathematical knowledge. In practice, they are

most often used synergistically and iteratively. The formal mathematical aspects of probability theory and probability distributions are not addressed

comprehensively herein. If a geotechnical perspective on probability theory is sought, the interested

reader is referred, for instance, to Ang & Tang (1975), Smith (1986), Harr (1987), Réthâti (1988) or

Baecher & Christian (2003). Here, it is perhaps useful to

distinguish, though in extreme synthesis, their respective contributions

in an applicative perspective. Statistical theory encompasses a broad range of topics. Among these, descriptive statistics deals with

the representation of the variability in data in a conventional form and with the fitting of probability

distribution functions to sample data, while the goal of inferential statistics is the modelling of patterns

in the data accounting for randomness and uncertainty in order to draw general inferences about the

variable parameter or process. In other words, the descriptive perspective is used to describe as well as possible a particular data set

(source set), with a view to interpolating within the data set. An inferential approach is required when it

is of interest to investigate the value and variability of random properties of a set (target set) which can

be expected to be in some way similar to the source set, but for which no or little quantitative information

is available a priori. In the inferential perspective, probability theory provides the framework by which

the outputs of statistical analysis can be processed consistently to provide a rational assessment of the

propagation and the effects of uncertainty with reference to a specific problem. Figure 3 provides a visualisation of the descriptive and inferential statistical approaches in the context

of geotechnical engineering. In descriptive approaches, the source site (i.e. where measurements are

available) coincides with the target site, and it is of interest to estimate properties at unmeasured locations.

In an inferential approach, results of analysis performed on data from the source site are applied to a

target site different from the source site, and where no

measurements are available.

2.2 Frequentist vs. degree of belief: the dual nature of probability

Probability theory is useful in modelling the observed behaviour of a variable parameter if a set of

measurements are available. However, an assumed behaviour can also be modelled. In the first case,

Figure 3. Descriptive and inferential statistical approaches in geotechnical engineering.

the frequentist nature of probability is referred to; in the second case, the degree of belief approach is

pursued.

Vick (2002) and Baecher & Christian (2003) provide essays on the dual nature of probability. An

important distinction between the two lies in the fact that estimates are essentially objective (i.e. obtained

through repeatable numerical procedures) in the first case and mostly subjective (i.e. relying on expert

knowledge and judgment) in the second.

Quantitative assessment of soil variability modelling requires use of descriptive and inferential statis

tics, as well as probabilistic modelling to process data from laboratory or in-situ measurements. Hence,

the paper addresses probabilistic techniques in a frequentist perspective.

The phases leading to the probabilistic modelling of a random variable are shown schematically in

Figure 4. The descriptive analysis includes the calculation of sample moments and (according to good

practice) visual inspection of data and histograms.

Inferential analysis includes the selection of a distribu

tion type, the estimation of distribution parameters and goodness-of-fit testing of the resulting distribution

to the source data. The dashed lines indicate that the results of the descriptive analysis can be used in the

inferential analysis; however, inference could also be performed without prior statistical description.

Descriptive and inferential statistical approaches to probabilistic modelling are addressed hereinafter

in an applicative perspective. The following treatment of the subject in no way intends to provide

a comprehensive overview of the underlying statistical and probabilistic theoretical frameworks. The

interested reader is referred to, for instance, the textbooks by Ang & Tang (1975) and Baecher &

Christian (2003).

2.3 Descriptive analysis

Any quantitative geotechnical variability investigation must rely on sets (in statistical terms, samples) of

measured data which are limited in size and quality. Hence, it is necessary to refer to sample statistics.

Sample statistics are imperfect estimators of the “real” population parameters. Hence, they are never

completely representative of the real distribution of the data, and are biased to some degree. Statements

regarding the ability of estimators to approximate the true population parameters can be made on the

basis of statistical theory.

The term sample statistic refers to any mathematical function of a data sample. For most engineering

purposes, sample statistics are more useful than the comprehensive frequency distribution (as given by

the histogram, for instance). An infinite number of sample statistics may be calculated from any given

data set. For the practical purpose of inferential

modelling, however, it is usually sufficient to calculate

the first four statistical moments of a sample, i.e. the mean, variance, skewness and kurtosis. Higher

moments are unreliable when estimated from most practical sample sizes.

The sample mean, i.e. the mean of a sample $\psi_1, \dots, \psi_n$ of a random variable is given by

Figure 4. Integrated descriptive and inferential analysis for probabilistic modelling of a random variable.

The sample variance of a set of data is the square of the sample standard deviation of the set itself. The

latter is given by

The unbiased estimates of the skewness and kurtosis of a data set are given by, respectively:

The sample moments as defined above are unbiased estimators of the distribution moments of the random

variable itself (i.e. the real values, calculated on a sample of infinite size).

2.3.1 Statistical estimation error

Any data set is unable to represent a population perfectly because of its size which is always limited in

practice. The sample statistics which are calculated from a data set are thus unable to represent the true

statistics of the population perfectly, in the sense that: (a) they may be biased; and (b) that there is some

degree of uncertainty in their estimation. Sample statistics, in other words, are affected by statistical

estimation uncertainty.

If a data set consisting of n elements is assumed to be statistically independent, the expected value

of the sample mean is equal to the (unknown) population mean; hence, the sample mean is an unbiased

estimator of the population mean. However, the sample mean has a variance and, consequently, a standard

deviation. The latter is given by $s/\sqrt{n}$, in which s is the sample standard deviation. The coefficient of

variation of statistical estimation error is thus defined as

Confidence intervals for the mean can also be calculated (see e.g. Ang & Tang 1975). It is important to

note that statistical estimation error decreases with increasing sample size.

2.4 Inferential analysis

The primary goal of inferential analysis is the modelling of a random variable (for which at least one

sample is available) by a probability distribution function. Such function assigns a level of probability

to every interval of the possible values taken by the random variable of interest. Once the type and the

parameters of probability distribution have been assigned, the probabilities associated with the interaction

of random variables in complex geotechnical systems (e.g. reliability analysis) can be calculated.

A distribution is called discrete if the random variable can only attain values from a finite set. A

distribution is called continuous if the range of the random variable is continuous, i.e. the variable can

take any value within a specific interval.

Probability distributions, in principle, may be more or less complex. However, it is advisable, for

practical purposes, to refer to probability distributions whose properties are well known. Moreover,

experience has shown that a relatively limited set of mathematical functions are able to fit satisfactorily

a wide range of observed or assumed distributions (Baecher & Christian 2003).

2.5 Probability distributions

Some among the most commonly used distributions in the geotechnical engineering literature are

addressed briefly in the following.

2.5.1 Uniform distribution

The probability density function of the uniform distribution, in which the minimum and maximum

bounds $[a]$ and $[b]$ are the parameters, is given by

Figure 5 shows two uniform distribution functions.

2.5.2 Triangular distribution

The probability density function of a triangular distribution with location a , scale b and shape c is

given by

A triangular distribution is correctly defined only for $\psi > a$ and $c < b$. Figure 6 shows two uniform

distribution functions. The mean and standard deviation of a triangular distribution are given by,

respectively:

Figure 5. Uniform distribution functions. Figure 6. Triangular distribution functions.

Triangular distributions are useful in cases where the upper and lower limits are known, and a most

probable value (mode) can be identified.

2.5.3 Normal distribution

The normal (or Gaussian) distribution has a probability density function given by

in which the parameters of the distribution, μ and σ , are the mean and standard deviation, respectively, of

the variable ψ . The normal distribution is defined in the range $-\infty < \psi < \infty$, and is symmetric around

the mean value. While it is one of the most frequently referred to distributions because of its many

important properties, it allows for negative values. As most physical properties cannot take negative

values, this can provide inconsistencies. Figure 7 shows two normal distribution functions.

2.5.4 Lognormal distribution

A random variable is lognormally distributed if its natural logarithm is normally distributed. The

probability density function of a lognormal distribution with parameters $[\lambda]$ and $[\xi]$ is

and is defined in the range $0 < \psi < \infty$. The distribution parameters, which represent, respectively, the

standard deviation and mean of the underlying normal distribution, can be obtained from the mean and

standard deviation of the variable ψ :

Figure 8 shows two lognormal distribution functions. The lognormal distribution is employed very

frequently because it is consistent with the fact that most physical properties are non-negative.

Figure 7. Normal distribution functions. Figure 8. Lognormal distribution functions.

2.5.5 Type-I Pearson beta distribution

A reverse-U type-I Pearson beta distribution is completely identified by four parameters: mean $[\mu]$;

standard deviation $[\sigma]$; lower bound $[a]$ and upper bound $[b]$. The probability density function is

in which

The characteristic parameters of the distribution are given by

in which

It must be verified that $\lambda_1 > 0$ and $\lambda_2 > 0$ for a type-I distribution to be defined. Figure 9 shows two

type-I beta distribution functions.

The type-I beta distribution is able to represent physical properties because it defines lower and upper

bounds and allows for skewness in the probability density function. In principle, such distribution should

be used for physical properties as it allows for skewness and upper and lower limits on possible values.

However, as shown in § 2.14, normal and lognormal distributions are used more frequently. This is

acceptable if the probability of negative values and/or very high values is very small.

2.6 Selection of distribution

The selection of a probability distribution to suitably represent a data set can be made using a variety of

approaches and techniques. In the following, the principle of maximum entropy and Pearson's moment

based system are addressed briefly.

Figure 9. Type-I beta distribution functions.

Table 2. Maximum-entropy criteria and constraints for the selection of a probability distribution.

Mean	Variance	Negative values	Upper and lower bounds	Maximum entropy distribution
Unknown	Unknown	Acceptable	Known	Uniform
Known	Known	Acceptable	Unknown	Normal
Known	Known	Unacceptable	Unknown (upper)	Lognormal
Known	Known	Acceptable	Known	Type-I beta

2.6.1 Principle of maximum entropy

In the context of the selection of a probability distribution, the more general principle of maximum

entropy (Jaynes 1978) states that the least biased model that encodes the given information is that which

maximises the entropy (i.e. the uncertainty measure) while remaining consistent with this information. By choosing to use the distribution with the maximum entropy allowed by available information, the

most appropriate distribution possible is selected. To choose a distribution with lower entropy would

imply unmotivated assumption regarding unavailable information; to choose one with higher entropy

would violate the constraints of the available information. Table 2 provides an example of the various criteria and constraints which may be used in selecting a

probability distribution using the maximum entropy principle. For example, according to the principle

of maximum entropy, if only the mean and the standard deviation of a random variable are known, and

negative values are not acceptable even for small probability levels, a lognormal distribution should

be selected. If, for instance, the minimum, maximum, mean value and standard deviation of a random

variable are known, the probability distribution to be adopted is a Pearson type-I beta distribution.

2.6.2 Pearson's moment-based system

Pearson developed an efficient system for the identification of suitable probability distributions based

on third and fourth-moment statistics (i.e. skewness and kurtosis, respectively) of a data set. Figure 10

shows Pearson's diagram, whose abscissas are given by, respectively (e.g. Rétháti 1988):

In principle, it may be seen in Figure 10 that the normal, exponential and uniform distribution are limiting

cases of the reverse-U type I beta distribution; these occupy single points in the (β_1 , β_2) diagram.

2.7 Distribution fitting

The fitting of a distribution to a data set can rely upon a variety of methods such as fitting by moments or

maximum likelihood. In moment fitting, the first four moments of the data set (mean, variance, skewness

and kurtosis) are calculated, and a distribution of the selected type - having the same corresponding

moments - is adopted (e.g. Ang & Tang 1975). The reader is referred to Ang & Tang (1975) and Baecher

& Christian (2003) for an insight into maximum likelihood approaches.

Figure 10. Space of Pearson's probability distributions

(from Rétháti 1988). Figure 11. Histogram and fitted log normal distribution for residual friction angle of a Canadian laminated clay shale (El-Ramly et al. 2003).

Figure 11 (El-Ramly et al. 2003) shows an example of a lognormal distribution fitted to the probability

histogram of residual friction angle measured from 80 drained direct shear tests for a laminated clay

shale at the Syncrude Tailings Dyke in Canada.

2.8 Hypothesis testing

Once a distribution has been fitted, it is advisable to verify its conformity to the data set by testing

its goodness-of-fit to the available data. A number of approaches to distribution testing are available:

these include, for instance, visual inspection of probability plots and hypothesis testing. While the first

approach relies on subjective assessment, hypothesis testing is based upon the calculation of appropriate

statistics and subsequent comparison with critical values for a reference confidence level.

The normality of residuals, for instance, may be tested

statistically using a wide range of normality

tests; the reader is referred to the statistical literature. Among the many available methods for testing

normality, the Wilk-Shapiro test (Shapiro & Wilk 1965) has been observed to provide good results in

comparison with other more widely used tests such as the Kolmogorov-Smirnov test, and is therefore

recommended (e.g. Thode 2002). The lognormality of a set of data can be tested for by applying normality

tests to the logarithms of the data values.

2.9 Second-moment probabilistic modelling

In second-moment approaches, the uncertainty in a random variable can be investigated through

its first two moments, i.e. the mean (a central tendency parameter) and variance (a disper

sion parameter). Higher-moment statistics such as skewness and kurtosis are thus not addressed.

Second-moment descriptive and inferential modelling of soil parameters are widely used in the

geotechnical literature because of their efficiency in transmitting important properties of data sets. Table 3. Multiplicative coefficient for the estimation of the standard deviation of a normally distributed data set with known range (e.g. Snedecor & Cochran 1989).
n N n n N n n N
n 11 0.315 30 0.244 2 0.886 12 0.307 50 0.222 3 0.510 13
0.300 75 0.208 4 0.486 14 0.294 100 0.199 5 0.430 15 0.288
150 0.190 6 0.395 16 0.283 200 0.180 7 0.370 17 0.279 8
0.351 18 0.275 9 0.337 19 0.271 10 0.325 20 0.268 The
sample coefficient of variation is obtained by dividing the
sample standard deviation by the

sample mean. It provides a concise measure of the relative dispersion of data around the central tendency

estimator:

The coefficient of variation is commonly used in geotechnical variability analyses. The advantages are

that it is dimensionless and provides a more physically meaningful measure of dispersion relative to the

mean. Coefficients of variation of the same physical properties at sites worldwide vary within a relatively

narrow range; moreover, they are thought to be independent of the geological age of the soil. This is

advantageous as it allows use of literature values with some confidence even at sites for which little or

no data may be available (Phoon & Kulhawy 1999a). Harr (1987) provided a "rule of thumb" by which

coefficients of variation below 10% are considered to be "low", between 15% and 30% moderate", and

greater than 30%, "high".

2.10 Approximate estimation of sample second-moment statistics

It may be necessary, at times, to obtain sample statistics in cases in which other statistics are known but

the complete data set is not available. A number of techniques yielding quick, approximate estimates of

sample statistics have been proposed.

2.10.1 Approximate estimation of sample standard deviation

For data which can be expected (on the basis of previous knowledge) to be symmetric about its central

value, the mean can be estimated as the average of the minimum and maximum values; hence, knowledge

of the extreme values would be sufficient. If the range and sample size are known, and if the data can be

expected to follow at least approximately a Gaussian (normal) distribution, the standard deviation can

be estimated by Eq. (23) (e.g. Snedecor & Cochran 1989), in which the coefficient N_n depends on the

size of the sample as shown in Table 3.

2.10.2 Three-sigma rule

Dai & Wang (1992) stated that a plausible range of possible values of a property whose mean value and

standard deviation are known can vary between the mean plus or minus three standard deviations. If it is

of interest to assign a value to the standard deviation, the statement can be inverted by asserting that the

standard deviation can be taken as one sixth of a plausible range of values. The three-sigma rule does

not require hypotheses about the distribution of the property of interest even though its origin relies on

the normal distribution. In case of spatially ordered data, Duncan (2000) proposed the graphical three-sigma rule method,

by which a spatially variable standard deviation can be estimated. To apply the method, it is sufficient

to select (subjectively or on the basis of regression) a best-fit line through the data. Subsequently, the

minimum and maximum conceivable bounds lines should be traced symmetrically to the average line. (a) (b)

Figure 12. Applications of the graphical three-sigma rule for the estimation of the standard deviation of: (a) undrained

shear strength; and (b) preconsolidation stress of San Francisco Bay mud (Duncan 2000).

The standard deviation lines can then be identified as those ranging one-third of the distance between

the best-fit line and the minimum and maximum lines. Applications of the graphical three-sigma rule

(Duncan 2000) are shown in Figure 12.

While the three-sigma rule is simple to implement and allows exclusion of outliers, Baecher &

Christian (2003) opined that the method may be significantly unconservative as persons will intuitively

assign ranges which are excessively small, thus underestimating the standard deviation. Moreover, for

the method to be applied with confidence, the expected distribution of the property should at least be

symmetric around the mean, which is not verified in many cases.

2.11 Correlation between random variables

When dealing with more than one random variable, uncertainties in one may be associated with uncer

tainties in another, i.e. the uncertainties in the two variables may not be independent. Such dependency

(which may be very hard to identify and estimate) can be critical to obtaining proper numerical results

in engineering applications (e.g. Ang & Tang 1975).

The most common measure of dependence among random variables is the correlation coefficient.

This measures the degree to which one uncertain quantity varies linearly with another.

The correlation coefficient for two uncertain quantities X and Y (represented respectively by two sets

of data $x_1 \dots x_n$ and $y_1 \dots y_n$) is defined as the ratio of the covariance of the two variables to the product

of the standard deviations of the two sets:

in which $m_{\psi 1}$ and $m_{\psi 2}$ are the sample means of 1 and 2, respectively. The correlation coefficient

is non-dimensional, and varies in the range $[-1,+1]$. A higher bound implies a strict linear relation

of positive slope (e.g. Figure 13a), while the lower bound attests for a strict linear relation of negative

slope. The higher the magnitude, the more closely the data fall on a straight line. The clause of linearity

Figure 13. Examples of correlation coefficient between two

variables.

should not be overlooked when interpreting the meaning of correlation: two uncertain quantities may be

deterministically related to one another, but may have negligible correlation if the relationship is strongly

non-linear (e.g. Figure 13d).

2.12 First-order second-moment (FOSM) approximation

In engineering design it is common to use parameters which are functions of measured properties. If

design is performed in non-deterministic perspective, it is necessary to estimate the uncertainty in such

parameters. This can be achieved by using techniques which process the uncertainty in source (measured)

parameters and in the transformation models used to derive the design parameters themselves. The type and complexity of the techniques to be used in uncertainty-based derivation of design

parameters depend mainly upon the level of accuracy required in the design phase and the approach by

which uncertainty in input parameters is modelled. It is intuitive that the sophistication of uncertainty

characterisation in output variables cannot exceed that of input variables; conversely, it is to be expected

that additional uncertainty would be introduced because of uncertainties in the engineering models used

to obtain design parameters, as well as by approximations in the uncertainty propagation techniques. If the uncertainties in measured parameters are modelled probabilistically in the second-moment

sense, it is necessary to adopt an uncertainty propagation technique which is compatible with random

variables described in such a way. Techniques such as Monte Carlo simulation, for instance, would

require assumptions regarding the probability distributions

of the measured parameters; this, however, is not compatible with the second-moment approach, in which it is chosen to represent random variables by only the first two statistical moments of the distributions, thereby neglecting all additional information. First-order second-moment approximation (FOSM) provides an effective means of investigating the propagation of second-moment uncertainties by providing an approximate estimate of the central tendency parameter (e.g. mean) and the dispersion parameter (e.g. standard deviation) of a random variable which is a function of other random variables (e.g. Ang & Tang 1975; Melchers 1999). By FOSM approximation, errors are introduced because of the fact that higher-order terms in the Taylor series are neglected. In engineering applications making use of second-moment statistics, it should be verified that the magnitude of such errors is small in comparison with the imprecision inherent to second moment modelling of input parameters and those associated with the transformation model. This is especially important in geotechnical applications, as the coefficients of variation of soil parameters are much larger than those of construction materials routinely used in structural engineering, e.g. concrete and steel. As an example of FOSM approximation, the calculation of the cone factor Uzielli et al. (2006) based on undrained strength data from triaxial compression tests and on net cone resistance from CPTU tests is shown in Figure 14. The figure shows, at each of the 17 reference measurement depths for which both sets of data were available: (a) mean values and standard deviations for undrained strength; (b) mean values and standard deviations for corrected cone resistance; and (c) mean values and standard

deviations for the cone factor. It can be seen that all expected values of the FOSM-estimated cone factor

are included within one standard deviation of the mean value in the soil unit. One very important application of FOSM techniques in the geotechnical literature is the formulation

of total uncertainty models, which allow second-moment description of random variables for direct

implementation in probabilistic design. It has been explained in §1.4 that the main contributing sources

to total uncertainty in a geotechnical design parameter calculated from a set of measured parameters

using an empirical, semi-empirical or theoretical transformation model are: (a) the inherent variability

of the measured parameters; (b) measurement error in the measured parameters; (c) statistical estimation

uncertainty in the measured parameters; and (d) bias and uncertainty in the transformation model.

Figure 14. First-order second-moment characterisation of cone factor: (a) triaxial compression (CAUC) - measured

undrained shear strength; (b) CPTU net cone resistance; (c) cone factor (Uzielli et al. 2006).

If the measured parameters are modelled in the second-moment sense (i.e. in mean and variance of

measurements are calculated), FOSM approximation can be used to transpose such statement to a quanti

tative framework, i.e. to define a second-moment total uncertainty model. A number of second-moment

uncertainty models have been proposed in the geotechnical literature. Phoon & Kulhawy (1999b), for

instance, proposed an additive model for the total coefficient of variation of a point design parameter

P_D obtained from a single measured property P_M using a transformation model M_t :

in which $COV(P, D)$ is the total coefficient of variation of the design property (neglecting variance reduc

tion effects due to spatial averaging); $COV_{\omega}(P, M)$ is the coefficient of variation of inherent variability

of the measured property; $COV_m(P, M)$ is the coefficient of variation of measurement uncertainty of the

measured property; and $COV_M(M, t)$ is the coefficient of variation of transformation uncertainty of the

transformation model. The interested reader is referred to Ang & Tang (1975) and Phoon & Kulhawy

(1999b) for an insight into the application of FOSM techniques for the derivation of total geotechnical

uncertainty models.

2.13 Practical difficulties in separating uncertainty components

A vast body of second-moment statistics of geotechnical properties are available in the literature. These

data generally pertain to sets of measurements obtained on soil units which were considered to be

homogeneous in some way (e.g. "clayey soils").

It is extremely difficult in practice to evaluate the various sources of uncertainty separately. Con

sequently, the coefficients of variation of measured parameters usually reported in the geotechnical

literature, even though they are termed "coefficient of variation of inherent variability", refer to total

uncertainty, i.e. they account for spatial variability, measurement error and statistical estimation error;

hence, their values generally overestimate the real magnitude of spatial variability alone. Phoon & Table 4. Second-moment sample statistics for water content of Edmonton clay (Fredlund & Dahlman 1971). Mean depth (m) No. of samples Mean(%) Std. dev.(%) COV_{ω} 0.6 392 27.4 6.0 0.22 1.2 430 28.9 5.1 0.18 1.8 439 31.2 4.1 0.13 2.4 422 33.0 4.1 0.12 3.0 415 32.8 3.9 0.12 3.6 392 34.7 3.5 0.10

4.2 362 35.0 4.3 0.12 4.8 307 35.5 4.3 0.12 5.4 249 35.5
4.6 0.13 6.0 177 34.8 4.5 0.13

Kulhawy (1999a) defined the coefficient of variation of inherent variability more rigorously as described

in §4.8. The calculation of such parameter requires procedures described in later sections of this paper.

Very few estimates of the “real” coefficient of variation of inherent variability are available in the

literature; these are also reported in §4.8. If data referring to total uncertainty from a source site are to be used inferentially, it must be verified

that it is appropriate to export data to the target site. The effects of exogenous factors, which may influence the magnitudes of a given geotechnical param

eter, should be recognised if pertinent. For instance, the undrained strength of a cohesive soil generally

varies with depth (even if the soil is compositionally homogeneous) due to increasing overburden stress,

overconsolidation effects and other in-situ factors. If the mean and standard deviation of undrained

strength are calculated for two soil units which are similar in composition and in stratigraphic location (i.e.

their upper interfaces are approximately at the same depth from ground surface), but different in thickness,

it may be expected that the mean value of the thicker layer will be greater than that of the other layer. Such possible bias is related to the existence of spatial trends in data. The issue of spatial trends will

be addressed in detail in §3.3. Total variability data from a source site should not be exported to a target

site unless the exogenous factors which may affect a soil property of interest are comparable at the two

sites. Alternatively, it could be useful and appropriate, in the above example, to address second-moment

statistics of stress-normalised strength to eliminate the

dependency on the exogenous factor “stress”. A number of warnings against uncritical exportation of total uncertainty data have been made (e.g.

R  th  ti 1988; Lacasse & Nadim 1996; Phoon & Kulhawy 1999a). For instance, extreme values and

outliers should not be accepted uncritically, as these could strongly bias statistical parameters. Outliers

can be identified through statistical procedures such as filtering; however, it is essential that data are

also inspected in the light of geotechnical knowledge. The temporal distribution of the collection of

data is important. If the data are collected over a time period of 1-2 weeks, the physical characteristics

of the soils may be regarded as time-invariant. Over longer periods of time, the scatter in properties

of interest may be partly attributed to changes occurring in soils through time. This is particularly true

for surficial soil layers, which are most exposed to meteorological variations and to soils in the zone

of fluctuation of groundwater level). Also, the standard deviation (and, consequently, the coefficient of

variation) generally increases with increasing dimensions of the sampling domain (e.g. areal extension

of a site; thickness of a layer). Therefore, it should be verified that the dimensions of the target site are

comparable to those of the source site.

2.14 Statistics of soil properties: literature review

The results of second-moment modelling of soil properties are generally provided in tabular form. The

compilation of descriptive statistics is not an entirely mechanical procedure. Geotechnical expertise is

very important in planning a statistical analysis as to obtain results which may be as significant and

useful as possible for characterisation and design. Given the above considerations, it is deemed useful

to report two examples of best-practice statistical processing of data from the geotechnical literature.

Subsequently, the results of a literature review of second-moment statistics are provided.

2.14.1 Second-moment statistics: best-practice examples

Fredlund & Dahlman (1971) provided tabulated values of second-moment sample statistics for Edmonton

clay. In their results for water content (Table 4), they subdivided the soil unit into sub-units of thickness Table 5. Number of laboratory tests of soils at Szeged (adapted from Rétháti 1988). Soil type Sample type w I P , I C e, S r , γ q u S1 T1 78 64 - - T2 56 37 54 19 T3 24 8 25 21 S2 T1 50 47 - - T2 85 44 85 29 T3 9 7 9 6 S3 T1 371 340 - - T2 531 281 538 227 T3 46 27 46 35 S4 T1 375 356 - - T2 512 197 513 299 T3 129 70 129 108 S5 T1 189 186 - - T2 52 9 52 32 T3 108 57 108 85

Table 6. Second-moment sample statistics of Szeged soils (adapted from Rétháti 1988). Soil type w (%) w L (%) w P (%) I P (%) I C e S r γ (kN/m³) q u (kPa)

Mean S1 31.1 44.5 24.2 20.6 0.62 0.866 0.917 18.8 144 S2 22.3 32.3 19.4 13.0 0.82 0.674 0.871 19.7 125 S3 24.5 32.2 20.7 11.5 0.63 0.697 0.902 19.7 173 S4 28.3 54.0 24.2 29.9 0.86 0.821 0.901 19.3 219 S5 28.6 52.8 25.4 27.4 0.82 0.823 0.905 19.2 184

COV S1 0.30 0.39 0.34 0.57 0.49 0.26 0.10 0.07 0.45 S2 0.18 0.13 0.12 0.37 0.37 0.13 0.12 0.04 0.38 S3 0.15 0.13 0.11 0.36 0.47 0.13 0.11 0.04 0.59 S4 0.17 0.21 0.14 0.34 0.20 0.11 0.10 0.03 0.53 S5 0.17 0.27 0.15 0.44 0.32 0.12 0.10 0.03 0.54

0.6 m. It is possible to appreciate the differences in mean values and coefficients of variation among

the sub-units. If the 6-metre layer had been treated as a whole, some variability information would have

been lost. The high numerosity of the samples, which reduces the statistical estimation error, should also

be noted.

In a comprehensive statistical study of the subsoil of the town of Szeged in Hungary performed in 1978,

Rétháti and Ungár (see Rétháti 1988) obtained second-moment statistics for soil physical characteristics

for 5 soil types (S1: humous-organic silty clay; S2: infusion loess, above groundwater level; S3: infusion

loess, below groundwater level; S4: clay; S5: clay-silt) using the results of approximately 11000 tests

from 2600 samples collected over 25 years.

The number of laboratory tests was reported (Table 5) by sample types (T1: partly disturbed samples

placed in metal boxes; T2: core samples taken from small diameter boreholes; T3: core samples taken

from large diameter boreholes) and the physical characteristics tested (water content w , plasticity index

I_P , consistency index I_C , void ratio e , degree of saturation S_r , unit weight γ , unconfined compression

strength q_u). Second-moment statistics are shown in Table 6. Recent compilations of soil statistics include

Sillers & Fredlund (2001) and Chin et al. (2006).

2.14.2 Second-moment statistics: literature review

The results of a literature review of second-moment statistics in the form of mean value, coefficient of vari

ation and coefficients of correlation between properties are reported hereinafter, as well as discussions

on suitable probability distributions for a number of soil properties.

The data presented hereinafter are useful for reference purposes; they should not be used uncritically

in design because statistics of those geotechnical parameters which are related to in-situ state are signifi

cantly dependent upon the in-situ state (or simulation

thereof in the laboratory). For these parameters,

it is difficult to identify typical values. Also, in geotechnical engineering it is often possible to measure

the same parameter using two or more testing methods and/or procedures. Different testing procedures

Figure 15. Coefficient of variation vs. mean values for laboratory measurements: (a) undrained shear strength; (b)

friction angle; (c) natural water content; (d) liquid and plastic limits; (e) unit weight; and (f) liquidity and plasticity

indices (Phoon & Kulhawy 1999a).

are generally characterised by different testing uncertainty. Moreover, using more than one testing

procedure will result in different measured values because the measurement occurs in a different way,

e.g. measurement of undrained strength by triaxial compression testing involves different deformations in

soils than, for instance, direct shear testing, triaxial extension or vane testing. Hence, the testing method

should be specified when reporting statistics from a source site. Lastly, it is generally not possible to

evaluate the degree of homogeneity in the soil units from which the statistics are calculated. If such

information is not provided, descriptive and inferential statistics will be misleading. Figure 15 (Phoon & Kulhawy 1999a) reports the results of an extensive literature review of coefficients

of variations of inherent variability for some laboratory-measured geotechnical properties, namely: (a)

undrained shear strength; (b) friction angle; (c) natural water content; (d) liquid and plastic limits; (e)

unit weight; and (f) liquidity and plasticity indices. In the diagrams, coefficients of variation of inherent

variability are plotted against mean value. Each point

represents a data set.

Figure 16. Coefficient of variation vs. mean values for in-situ measurements: (a) cone tip resistance from CPT; (b)

undrained shear strength from field vane testing; (c) SPT blow counts; (d) A and B readings from DMT testing; (e)

pressuremeter limit pressure; (f) dilatometer material index; (g) dilatometer horizontal stress index; (h) dilatometer

modulus and pressuremeter modulus (Phoon & Kulhawy 1999a).

Figure 16 reports the coefficients of variations of inherent variability for some in-situ testing param

eters (Phoon & Kulhawy 1999a): (a) cone tip resistance from CPT; (b) undrained shear strength from

field vane testing; (c) SPT blow counts; (d) A and B readings from DMT testing; (e) pressuremeter limit

stress; (f) dilatometer material index; (g) dilatometer horizontal stress index; (h) dilatometer modulus

and pressuremeter modulus.

Table 7. Approximate guidelines for second-moment statistics of soil parameters.

Test type Property * Soil type Mean COV (%)

Lab strength s_u (UC) Clay 10-400 kN/m² 20-55 s_u (UU) Clay 10-350 kN/m² 10-30 s_u (CIUC) Clay 150-700 kN/m² 20-40 ϕ' Clay & sand 20-40 5-15

CPT q_T Clay 0.5-2.5 MN/m² <20 q_c Clay 0.5-2.0 MN/m² 20-40 Sand 0.5-30.0 MN/m² 20-60

VST s_u (VST) Clay 5-400 kN/m² 10-40

SPT N Clay & sand 10-70 blows/ft 25-50

DMT A reading Clay 100-450 kN/m² 10-35 Sand 60-1300 kN/m² 20-50 B reading Clay 500-880 kN/m² 10-35 Sand 350-2400 kN/m² 20-50 I D Sand 1-8 20-60 K D Sand 2-30 20-60 E D Sand 10-50 MN/m² 15-65

PMT p_L Clay 400-2800 kN/m² 10-35 Sand 1600-3500 kN/m²

20-50 E PMT Sand 5-15 MN/m² 15-65

Lab index w n Clay & silt 13-100% 8-30 w L Clay & silt
30-90% 6-30 w P Clay & silt 15-25% 6-30 I P Clay & silt
10-40% ** I L Clay & silt 10% ** γ , γ_d Clay & silt 13-20
kN/m³ <10 D R Sand 30-70% 10-40 *** 50-70 ****

Lab. consolidation C_c Not reported - 10-37 p'_c Not
reported - 10-35 OCR Not reported - 10-35

Not reported k Saturated clay - 68-90 Partly saturated clay
- 130-240

Not reported c_v Not reported - 33-68

Not reported n All soil types - 7-30 e All soil types -
7-30 e₀ All soil types - 7-30

* s_u = undrained shear strength; UC = unconfined
compression test; UU = unconsolidated-undrained tri

axial compression test; CIUC = consolidated isotropic
undrained triaxial compression test; ϕ' = effective

stress friction angle; q_T = corrected cone tip resistance;
q_c = cone tip resistance; VST = vane shear test;

N = standard penetration test blow count; A and B readings,
I_D, K_D and E_D = dilatometer A and B read

ings, material index, horizontal stress index and modulus;
p_L and E_{PMT} = pressuremeter limit stress and

modulus; w_n = natural water content; w_L = liquid limit; I
P = plasticity index; I_L = liquidity index; γ and

γ_d = total and dry unit weights; D_R = relative density; C_c
= compression index; p'_c = preconsolidation pres

sure; OCR = overconsolidation ratio; k = permeability
coefficient (direction not specified); c_v = coefficient

of vertical consolidation; n = porosity; e = void ratio; e₀
= initial void ratio. ** COV = (3-12%)/mean.

*** Total variability for direct method of determination.

**** Total variability for indirect determination using SPT
values. Table 7 reports typical ranges of mean values and
coefficients of variation (accounting for inher

I P -0.59 0.53 0.85 0.56 1.00

G S 0.32 -0.17 0.06 0.09 0.24 1.00

γ_d 0.75 -0.59 -0.61 -0.50 -0.58 0.51 1.00

w -0.85 0.70 0.64 0.48 0.66 -0.38 -0.93 1.00

S r -0.77 0.69 0.61 0.39 0.72 0.08 -0.51 0.75 1.00

c ' 0.37 -0.38 -0.10 0.00 -0.21 -0.06 0.26 -0.34 -0.28 1.00

ϕ' 0.72 -0.63 -0.68 -0.49 -0.75 0.18 0.82 -0.81 -0.65 0.10
1.00

* % gr. = percent gravel; % sd. = percent sand; w L =
liquid limit; w P = plastic limit; I P =plasticity index;

G S = specific gravity; γ_d = dry unit weight; w = water
content; S r = degree of saturation; c ' = effective

cohesion; ϕ' = effective friction angle.

Kulhawy & Trautmann (1996), Lacasse & Nadim (1996), Phoon &
Kulhawy (1999a) and Jones

et al. (2002).

2.15 Correlation in soil properties: literature review

The quantification of correlation between two or more soil
properties is important in the context of

probabilistic geotechnical approaches, as it provides a
more realistic assessment of uncertainty in design

parameters which are functions of the properties
themselves. A full literature review is hardly feas

ible. Literature values for the linear correlation
coefficient are site-specific and should not be exported

uncritically. Selected results are shown hereinafter
primarily to highlight the dependence of correlation

values from a number of factors. For instance, dependence
from soil type is well established. The testing

method used to obtain a given parameter generally affects
the numerical value of the parameter itself

(e.g. undrained strengths measured on the same soils by triaxial compression, direct shear or triaxial

extension are different because of the different failure mechanisms involved). Hence, it is to be expected

that, as in the case of sample statistics, correlation will also depend from the testing method to some

extent. It is also important that soil units from which statistics are to be calculated are homogeneous.

Magnan & Bagheri (1982) provided an exhaustive collection of correlation values.

Holtz & Krizek (1971) compiled correlation matrices for a number of soil properties of pervious

materials (primarily clean sands, gravels and boulders) and impervious materials (clayey gravels) from

the Oroville Dam area in California, measured by laboratory testing. Table 8 illustrates the correlation

matrix for impervious materials; Table 9 illustrates the correlation matrix for the combined data (pervious

and impervious materials). Comparing Tables 8 and 9, it is seen that in many cases the magnitude of

correlation varies significantly (in some cases the sign is even inverted): for instance, the correlation

Table 10. Correlation matrix for Szeged soil S1: humorus-organic silty clay (adapted from Rétháti 1988).

Parameter * w w L w P I P I C e S r γ

w 1.00

w L 0.61 1.00

w P 0.76 0.77 1.00

I P 0.33 0.89 0.39 1.00

I C -0.23 0.45 0.36 0.39 1.00

e 0.85 0.39 0.40 0.29 -0.36 1.00

S r -0.04 0.04 0.14 -0.02 0.05 -0.29 1.00

γ -0.44 -0.05 -0.09 -0.02 0.30 -0.58 0.34 1.00

Table 11. Correlation matrix for Szeged soil S2: infusion loess, above groundwater level (adapted from Rétháti

1988).

Parameter * w w L w P I P I C e S r γ

w 1.00

w L 0.06 1.00

w P 0.36 0.08 1.00

I P -0.13 0.86 -0.44 1.00

I C -0.59 0.20 0.33 0.02 1.00

e 0.72 0.01 0.00 0.01 -0.55 1.00

S r 0.62 0.00 0.22 -0.12 0.38 -0.04 1.00

γ -0.33 0.14 0.03 0.11 0.26 -0.85 0.46 1.00

Table 12. Correlation matrix for Szeged soil S3: infusion loess, below groundwater level (adapted from Rétháti

1988).

Parameter * w w L w P I P I C e S r γ

w 1.00

w L 0.26 1.00

w P 0.35 0.33 1.00

I P 0.07 0.87 -0.12 1.00

I C -0.69 0.29 0.11 0.24 1.00

e 0.65 0.05 0.00 0.05 -0.52 1.00

S r 0.46 0.17 0.18 0.09 -0.30 -0.26 1.00

γ -0.11 0.08 -0.03 0.10 0.10 -0.44 0.33 1.00

* w = water content; w_L = liquid limit; w_P = plastic limit; I_P = plasticity index; I_C = consistency index;

e = void ratio; S_r = degree of saturation; γ_d = dry unit weight.

between liquid limit and gravel fraction is 0.13 for the impervious materials, but becomes -0.52 if the

combined data are considered. Rétháti (1988) reported matrices of correlation coefficients for Szeged (see also §2.14.1) soil types S1

(Table 10), S2 (Table 11) and S3 (Table 12). If Tables 10, 11 and 12 are compared, it is seen that some of

the correlation values vary little between soil type (e.g. liquid limit vs. plasticity index). On the contrary,

a number of values are very different. Such is the case, for instance, for water content vs. liquid limit,

degree of saturation vs. consistency index and unit weight vs. void ratio. Even more significant is the

observation that there are substantial variations in correlation coefficients between S2 and S3, as these

soil units refer to the same soil type (infusion loess), and the distinction is only between the stratigraphic

depth with respect to groundwater level (S2 are above; S3 are below). While these variations can be

explained based on geotechnical considerations (i.e. the expectable difference in the degree of saturation

and void ratio between S2 and S3), it is important to remark that they are appreciable as a consequence of

a correct choice in the definition of soil units: if S2 and S3 had been classified solely on the basis of soil

type, only “lumped”, less informative and possibly unconservative correlation values for the infusion

loess could have resulted. For best-practice, if it is not possible to collect data and calculate the correlation coefficient using Eq.

(24), maximum care should be taken as to select literature values which: (a) pertain to soil units which

Figure 17. Dependence of probability distributions from in-situ state and soil type (Popescu et al. 1998).

are sufficiently homogeneous in terms of composition, in-situ state and mechanical behaviour; (b) refer

to soil types which are affine to the ones under investigation; and (c) are obtained using the same testing methods.

The correlation between effective cohesion and effective friction angle is perhaps the most widely

referred to in literature applications, as it enters important analyses such as foundation bearing cap

acity and slope stability. A comparative literature review yields the following ranges of values for

such correlation coefficient: $p = -0.47$ (Wolff 1985); $-0.49 \leq p \leq -0.24$ (Yucemen et al. 1973);

$-0.70 \leq p \leq -0.37$ (Lumb 1970); $p = -0.61$ (Cherubini 1997). In absence of specifically calculated

data, a parametric approach using $-0.75 \leq p \leq -0.25$ can be used for practical applications.

2.16 Probability distributions for soil properties: literature review

Different probability distribution models have been selected, even for the same soil property, by different

authors. This suggests that distributions are site and parameter-specific, and that there is no universally

“best” distribution for soil properties. In-situ effects, which may result in a spatial trend, may also be

relevant. For instance, based on cone penetration data from artificial and natural deposits, Popescu et al.

(1998) observed that the distribution of soil strength in shallow layers were prevalently positively skewed,

while for deeper soils the corresponding distributions tended to follow more symmetric distributions.

This is shown in Figure 17, which also reports the different distributions which were selected for the

various soil units.

Despite the substantial data set-dependence of best-fit probability distributions, a number of literature

results are reported in the following. Table 13. Results of Corotis et al. (1975) investigation on probability distributions (in %). N LN Other w 67 33 - w L 67 33 - w P - 100 - e 67 - 33 c v 67 33 - Table 14. Probability distributions for different soil properties (adapted from Lacasse & Nadim 1996). Soil property Soil type Distribution Cone resistance Sand LN Clay N/LN Undrained shear strength Clay (triaxial tests) LN Clay (index tests) LN Clayey silt N Stress-normalised undrained shear strength Clay N/LN Plastic limit Clay N Submerged unit weight All soils N Friction angle Sand N Void ratio, porosity All soils N Overconsolidation ratio Clay N/LN Corotis et al. (1975) investigated whether a number of properties of three groups of soils could be

described by the normal or lognormal distribution. The Kolmogorov-Smirnov goodness-of-fit test was

applied to data sets from the three soil types. The percentage of data sets to which the normal (N) or

lognormal (LN) distributions could be fit with sufficient confidence is shown in Table 13. Lacasse & Nadim (1996) reported the results of a review of probability distribution selection for some

soil properties; these are shown in Table 14. It should be noted that best-fit probability distributions may

also depend on soil type. As an example of probability distribution selection using Pearson's moment-based system, the results

reported in Rétháti (1988) for Szeged soils are shown in Table 15. The parameters β_1 and β_2 are calculated

using Eq. (20) and Eq. (21), respectively. The points are plotted on Pearson's chart (also shown in Figure

10) as illustrated in Figure 18. With reference to Figure 10 and Figure 18, a number of observations can be made: (a) the consistency

index is highly symmetric (i.e. has a small skewness), and can therefore be approximated for 3 of 5 soil

units using a normal distribution; (b) a distribution type of general validity cannot be adopted for void

ratio; (c) the degree of saturation is highly non-symmetric and can be approximated using a J-shaped

and a reversed U-shaped beta distribution. As a general observation, points corresponding to the same property for different soil units generally

plot in different areas of the chart; this reflects the influence of soil type and in-situ state on data

distribution. Hence, it is difficult to associate a specific probability distribution to a soil property a priori.

2.17 Probabilistic slope stability: literature examples

Second-moment probabilistic slope stability analysis allows assessment of the probability of a pre

defined type of failure. The reliability index can be calculated using a variety of approaches, such

as Monte Carlo simulation or First-Order Reliability Method (FORM). Overviews of second-moment

probabilistic slope stability investigations are provided in El-Ramly et al. (2002) and Nadim et al. (2005). Griffiths & Fenton (2004) conducted a probabilistic investigation on the simplified cohesive slope

model shown in Figure 19. Initially, they obtained the deterministic factor of safety from limit-equilibrium

methods as a function of the normalised, dimensionless undrained shear strength (Figure 20). The probabilistic approach made use of Monte Carlo simulation (essentially repeated realisations of

deterministic limit-equilibrium analyses using strength

values sampled from an assumed distribution, in

this case lognormal), and allowed to establish the relationship between deterministic factor of safety and

Table 15. β_1 and β_2 values for selecting probability distribution using Pearson's chart for the physical characteristics

of Szeged soils (adapted from Rétháti 1988).

Soil w w L w P I P I C e S r γ q u

S1 β_1 2.76 4.12 6.81 1.93 0.13 6.50 7.90 1.28 0.02 β_2
7.39 8.10 12.30 4.92 3.34 11.19 14.14 3.98 1.86

S2 β_1 0.01 0.74 0.34 0.49 0.02 0.01 0.86 0.09 0.94 β_2
3.45 3.43 3.27 2.69 3.17 2.67 3.39 3.87 3.93

S3 β_1 0.03 0.96 0.14 0.85 0.00 1.30 3.17 2.89 5.06 β_2
7.62 5.13 4.32 4.46 3.31 5.52 7.38 11.59 10.72

S4 β_1 0.05 0.34 0.13 0.64 0.03 0.13 3.20 1.06 4.80 β_2
7.17 3.19 3.47 3.57 4.61 4.03 7.35 8.14 10.95

S5 β_1 1.10 0.02 2.92 0.00 0.98 0.36 2.89 0.10 2.72 β_2
6.70 2.31 9.15 2.17 5.13 4.14 6.50 4.94 6.74

Figure 18. β_1 , β_2 value pairs for physical characteristics of Szeged soils plotted by soil unit (1-5) in Pearson's chart

(Rétháti 1988).

probability of failure (shown in Figure 21 for different values of the coefficient of variation of undrained

shear strength). A key to understanding the results lay in the observation that if the median value of

the lognormal distribution of normalised strength was smaller than 0.17, increasing the coefficient of

variation of the distribution (i.e. increasing standard deviation or decreasing mean) resulted in decreasing

probability of failure; on the contrary, for median values exceeding 0.17, probability of failure increased

with increasing coefficient of variation of the

distribution (Figure 22).

Nadim & Lacasse (1999) demonstrated the application of FORM to undrained slope stability analysis.

They evaluated the factor of safety and probability of failure of a slope (shown in Figure 23) consisting

of 2 clay layers with assessed large-scale average property distributions under static and seismic loading

(Table 16). Stability analysis relied on Bishop's method; hence, circular slip surfaces were identified.

Model uncertainty (i.e. bias and dispersion) was also modelled with second-moment parameters.

Application of FORM showed that the critical surface with the lowest deterministic safety factor is

not the critical surface that has the highest probability of failure. Slip surface 1, which cuts through

both soil layers, has the lowest deterministic safety factor. However, due to spatial averaging effects,

Figure 19. Cohesive slope test problem (Griffiths &

Fenton 2004). Figure 20. Relationship between factor of safety and normalised strength for the cohesive model slope using limit-equilibrium methods (Griffiths & Fenton 2004).

Figure 21. Probability of failure versus factor of

safety for the cohesive model slope (Griffiths & Fenton

2004). Figure 22. Probability of failure versus coefficient of variation for different values of the median of the lognormal distribution of normalised strength (Griffiths & Fenton 2004).

the uncertainty in the computed safety factor for slip surface 1 is significantly less than that for slip

surface 2, which only cuts through the top layer (Figure 24). Low (2003) proposed a spreadsheet-based methodology for first-order reliability method reliability

based slope stability evaluation using the generalised method of slices. The methodology allows second

moment formulation of soil properties (cohesion, friction angle and unit weight) and performs an iterative

numerical search for the minimum-reliability slip surface, which is generally different (as shown in the

previous example) from the minimum factor-of-safety surface. Figure 25 shows an example application

of the method.

3 SPATIAL CORRELATION ANALYSIS

Second-moment statistics alone are unable to describe the spatial variation of soil properties, whether

measured in the laboratory or in-situ. Two sets of measurements may have similar second-moment

Figure 23. Reference slope for FORM analysis by Nadim & Lacasse (1999). Table 16. Large-scale average property distributions for static and seismic conditions for FORM analysis (Nadim & Lacasse 1999). Variable Probability distribution Mean COV s_u in Layer 1 Lognormal 75 kPa 25% γ_t in Layer 1 Normal 17.5 kN/m³ 5% s_u in Layer 2 Lognormal 150 kPa 15% γ_t in Layer 2 Normal 17.5 kN/m³ 5% Model uncertainty, ϵ Normal 1.0 10%

Figure 24. Computed distribution of safety factors for slip surfaces 1 and 2 (Nadim & Lacasse 1999).

statistics (i.e. mean and standard deviation) and statistical distributions, but could display substantial

differences in spatial distribution. Figure 26 provides a comparison of two-dimensional spatial distribu

tion of a generic parameter ξ having similar second-moment statistics and distributions (i.e. histograms),

but different magnitudes of spatial correlation: weak correlation (top right) and strong correlation (bottom

right).

Knowledge of the spatial behaviour of soil properties is often paramount in geotechnical analysis

and design. Among the many reasons are: (a) geotechnical

design is based on site characterisation,

which objective is the description of the spatial variation of compositional and mechanical parameters

of soils; (b) the values of the parameters themselves very often depend on in-situ state factors (e.g. stress

level, overconsolidation ratio, etc.) which are related to spatial location; (c) for large-scale engineering

Figure 25. Spreadsheet search for reliability-based critical slip surface using the generalised method of slices

(Low 2003).

Figure 26. Comparative representation of spatial data with similar statistical distributions (top and bottom right) but

different magnitudes of spatial correlation: weak correlation (top right) and strong correlation (bottom right) (from

El-Ramly et al. 2002).

endeavours such as dams or roads, it is generally expected that heterogeneous site characteristics will be

revealed by investigations at spatially distant locations.

3.1 Modelling spatial variability

The spatial variation of a soil deposit in any direction could, in principle, be characterised in detail if a

sufficiently high number of measurements were taken. This, however, is impossible in practice, and could

even be recognised as superfluous. Thus, as stated by Baecher (1982), the randomness in the variation

of soil properties is often a convenient hypothesis to make as a result of this limitation in knowledge:

“There is clearly nothing random about the variation of soil properties from one location to another.

In principle, with a sufficient number of measurements, the

properties at every location could be known

within the accuracy of testing. Practically, however, testing is limited. Thus, it is convenient to model

the spatial variation of soil properties as if it were random, and to use associated statistical results to

quantify interpolation errors. Actually, such an approach need never make assumptions of randomness.

It need only recognize that the mathematics used to model random variations can also be used to

summarize spatial behaviour, and then exploit that similarity to apply the mathematical results to spatial

interpolation, averaging, or other problems.”

Structures explanations of the statistical techniques used for the investigation spatial variability are

provided by Priestley (1981) and Baecher & Christian (2003). While the former provides an exhaus

tive insight into the mathematical framework of time series analysis, the latter focuses specifically on

application of such techniques to geotechnical engineering.

3.2 Stationarity and data transformation

Statistical modelling of spatial variability relies heavily on the hypothesis of data stationarity. While the

implications and the methods to assess stationarity are addressed in some detail in Section 6, it may

Figure 27. Visual representation of decomposition in the one-dimensional case.

be sufficient, here, to assert that stationarity denotes the invariance of a data set’s statistics to spatial

location. More specifically, weak stationarity is deemed sufficient to allow application of statistical

techniques. If a data set of interest is not stationary, the results of statistical analyses may be erroneous or biased.

Hence, it is necessary to transform the data set. Data transformation is a general term referring to a number

of techniques (mostly from time series analysis) which purpose is the transformation of a non-stationary

data set to a stationary set. Decomposition is by far the most widely adopted data transformation technique

in the geotechnical engineering literature. Such technique is illustrated in some detail in the following.

Other data transformation techniques include differencing and variance transformation; a review of such

methods is provided in Jaksa (2006).

3.3 Decomposition

By decomposition, the spatial variability of a spatially ordered measured geotechnical property

$[\psi(z_1 \dots z_n)]$ in a sufficiently physically homogeneous (according to some user-defined criterion) soil

unit may be broken down into a trend function $[t(z_1 \dots z_n)]$ and set of residuals about the trend

$[\xi(z_1 \dots z_n)]$. In the one-dimensional case, for instance, taking depth (z) as the single spatial coordinate,

decomposition is expressed by the following additive relation:

An example is shown in Figure 27. Equation (26) neglects measurement error. This is justified by

the hypothesis that the aleatory uncertainty resulting from inherent soil variability and the epistemic

uncertainty due to measurement error are uncorrelated, and can be addressed separately. The presence of

a spatial trend in soil properties, even for extremely homogeneous soils, is very common in geotechnical

engineering, where the dependence of parameter magnitude from factors such as overburden stress and

stress history is well recognised. The decomposition procedure, as spatial variability analyses in general,

can be extended to higher-dimensional cases. Figure 28 (Przewłocki 2000) shows the decomposition of

soil resistance data in a two-dimensional spatial extension. Hereinafter, reference will be made primarily

to the one-dimensional case as this is most frequently encountered in practice and in the literature. In the decomposition procedure outlined above, the trend is described deterministically by an equation;

the residuals are characterised statistically as a variable, with (usually) zero-mean and (always) non-zero

variance.

Figure 28. Example of 2-D decomposition of cone resistance: (a) measured data and trend surface; (b) residuals of

detrending (Przewłocki 2000).

The decomposition procedure is arbitrary to some degree: first, the type of trend function (e.g.

polynomial, exponential, etc.) to be used is established by the user; second, the estimation of trend

parameters can be achieved using a variety of techniques.

The separation into a deterministic trend and random variation is thus an artefact of the analysis; there

is no univocally “correct” trend to be identified, but rather a “most suitable” one. The choice of the trend

function must be consistent with the requirements of the mathematical techniques adopted, and must,

more importantly, rely on geotechnical expertise.

3.4 Spatial trend functions

The least-squares method is widely used to estimate the parameters to be fit to a set of data and to

characterise the statistical properties of the estimates.

Trend removal by least-squares regression has

been utilised by several researchers in the geotechnical literature (e.g. Alonso & Krizek 1975; Baecher

1987; Campanella et al. 1987; Brockwell & Davis 1991; Kulhawy et al. 1992; Jaksa 1995; Jaksa et al.

1997; Phoon et al. 2003c; Wu 2003).

Regression analysis is useful in describing the relationship between two random variables (the soil

property and the spatial direction variable). Such relationship is described in terms of the mean and

variance of the first conditioned on the second. The main outputs of regression analysis are the regression

parameters (i.e. the parameters describing the deterministic trend) and the conditional variance of soil

property on the spatial coordinate. These account for, respectively, the assumed shape of the trend

function and the part of total spatial variability which may be ascribed to the trend itself.

Figure 29 provides intuitive schemes of several possible regression cases, related to the type of

relationship between variables (linear vs. non-linear) and the characteristics of data scatter around the

trend with depth (constant vs. non-constant conditional variance). Trends are shown as solid lines;

dispersion envelopes (e.g. trend $\pm$ conditional standard deviation) are shown as dashed lines.

Least-squares can be implemented in several versions: ordinary least-squares (OLS) relies on the

hypothesis of homoscedasticity (i.e. constant variance of the residuals) and does not assign weights

to data points; generalised least-squares (GLS) relaxes the hypothesis of homoscedasticity in favour

of an independent estimate of the variance of the

residuals, and allows for weighing of data points

as a consequence of the variance model. Linear and non-linear OLS can be seen as special cases of

GLS. Statistical regression is commonly performed using dedicated software. The reader is referred, for

instance, to Ang & Tang (1975) for theoretical aspects of regression procedures.

3.4.1 Uncertainty in estimated trend parameters

Decomposition brings the effect of attributing some of the spatial variation of a soil property to the trend

and some to the residuals. Even though the trend is a deterministic function, there is statistical estimation

uncertainty due to the limited size of the data sets used for trend identification.

Figure 29. Possible cases of least-squares regression. If a quantitative estimation of the spatial variability of soil properties is of interest, it is necessary

to quantify the magnitude of the components of such variability, i.e. the variability related to the trend

function and that pertaining to the set of residuals. Distinct approaches are required for the two compon

ents. Here, the estimation of the uncertainty in trend parameters is addressed. The uncertainty in the

spatial variability of the set of residuals will be examined subsequently. If the trend interpolates data values very closely, the variance of the residuals will decrease but the

uncertainty in the estimation of trend parameters will be greater. Conversely, a simpler trend will have a

smaller estimation uncertainty in its parameters, but the variance of the resulting set of residuals will be

larger, approaching infinity as the trend interpolates the data perfectly. Figure 30 provides a comparative

visual appreciation of the effects of removing a first and a

second-order polynomial from the same

source data set. The results of first-order detrending still show a well-defined spatial trend (and are

therefore unlikely to be stationary) and a larger variance than the residuals of second-order detrending,

which show no appreciable spatial trend and which could be stationary (verification requires formal

procedures, see Section 6).

3.4.2 Criteria for trend selection

While the formal compliance to the reference mathematical framework is necessary for the quantitative

assessment of variability, the importance of geotechnical engineering judgement must not be downplayed

because the presence of trends is usually motivated by geologic processes or other physical reasons which

should be recognised by the geotechnical engineer. To avoid inconsistency with the underlying physical

Figure 30. Comparative visual appreciation of the effects of removing a linear and a second-order polynomial

from the same data set: (a) source data set with superimposed first and second-order polynomials; (b) residuals of

first-order detrending; (c) residuals of second-order detrending.

reality of the model under investigation, trends should not be removed if they are in contrast with - or

are not amenable to - physical reasons (e.g. Akkaya & Vanmarcke 2003). In-situ overburden stress, for

instance, is a major factor contributing to the occurrence of spatial trends in soil masses. Alonso & Krizek

(1975) suggested that visual examination may provide the best means for trend identification. Baecher

& Christian (2003) stated that "trends should be kept as

simple as possible without doing injustice to a set of data or ignoring the geologic setting”.

Several numerical procedures for the assessment of the trend “quality” have been applied in the

geotechnical literature: Cafaro & Cherubini (2002), for instance, appraised the appropriateness of

removed trends by the CUSUM analysis technique (e.g. Cherubini et al. 2006). However, geotech

nical expertise should play a central role in the selection of the complexity level of the trend. A study by

Chiasson & Wang (2006) on the variability of sensitive Champlain marine clays provides an example of

decomposition and trend analysis in a geotechnical perspective.

3.4.3 Set-specificity of trend

One of the main benefits of inferential statistical analyses is the possibility, if a reasonably conspicuous

bulk of information is available from one or more sets, to make statements using about the stochastic

nature of other presumably similar sets. The convenience of decomposition and the attribution of a part

of the spatial variation to a deterministic trend have been discussed previously. It has been stated that

the correlation structure of the residuals is different from that of the original data - it shows less spatial

dependence and has reduced variance; the degree to which this occurs depends on the complexity of

the trend, which is established by the user as noted previously. The use of only the residuals from a

source site at the target site would lead to an unconservative underestimation of soil variability as the

uncertainty pertaining to the trend would not be addressed.

A trend removed with reference to the minimisation of variance for one specific data set may be

appropriate for one specific data set, but is generally not suitable for direct application to another set.

It is thus conceptually incorrect to apply the trend obtained for a specific data set to other data sets

(even if pertaining to the same geotechnical parameter) unless it has been verified that the criteria which

were used for trend estimation are sufficiently similar in the target set. Trends which are qualitatively

Figure 31. Example of set-specificity of spatial trend function.

associable to - and whose presumed associated level of uncertainty is similar to - the trend of the "source"

should be expected at the target site. Consider the example shown in Figure 31. Suppose that a boring log and a subsequent laboratory

investigation have provided a stratigraphic profile indicating the presence of two clay layers "A1" and

"A2" separated by a cohesionless layer. Atterberg limits measurements are performed on 4 samples

for both A1 and A2, indicating that the plasticity index I_P is very similar for the two layers. Triaxial

testing is performed on the samples as well, yielding the values of undrained shear strength shown on

the figure. The overconsolidation ratio suggests that A1 is more heavily overconsolidated than A2. The

trends identified for undrained shear strength s_u in A1 and A2 are very different. Despite the substantial

homogeneity of A1 and A2 in index properties and the fact that linear trends are selected for both layers

(and, hence, the magnitudes of the scatter of the residuals around the trends are presumably similar), it

would be grossly incorrect to apply the trend of one layer

to the other in the context of a spatial variability

analysis. In the context of soil variability modelling, non-exportability of trend functions is often referred to as

“site-specificity” in the geotechnical literature; it is argued here that such terminology is not exhaustive

and should be replaced by “set-specificity” because, as has been shown in the present example, distinct

trends may occur even at the same site and, possibly, even in the same soil unit (consider, for instance, the

variation of overconsolidation ratio with depth in highly homogeneous soils). Figure 32 illustrates the

results of decomposition on submerged unit weight data for Troll marine clay (Uzielli et al. 2006).

The difference in trends between the two physically homogeneous soil units (UN1 and UN2) is evident. It is thus paramount to address spatial variability analysis with constant and careful reference to a

geotechnical perspective; the latter should never be subordinated to mathematical convenience.

3.5 Residuals

The residuals of decomposition represent the part of spatial variability which cannot be explained by a

relatively simple function of the reference spatial coordinate. They are usually a zero-mean set which,

when plotted against the spatial coordinate, fluctuates around the mean value. A fundamental assumption inherent to decomposition and trend fitting is that the residuals of decom

position are spatially uncorrelated, i.e. their fluctuation is completely random. This is equivalent to

stating that the uncertainty in trend parameters accounts for the entire spatial structure of soil variability.

As the level of complexity of the trend is chosen by the user, it would be possible to select a trend

which would interpolate available data completely; however, the uncertainty in trend parameters would

correspondingly approach infinity. It is more convenient to exploit the main advantage of decompos

ition, i.e. the possibility to identify a simple trend and to model the set of residuals as a random variable

which may be effectively investigated using appropriate and well-established statistical techniques. In a

Figure 32. Decomposition and second-moment analysis for submerged unit weight of Troll marine clay: (a) original

data, trends and standard deviation; (b) residuals (Uzielli et al. 2006).

second-moment perspective, the part of spatial variability that remains after trend removal corresponds

to the variance of the residuals.

Investigation of the spatial pattern of the residuals is a central phase of variability modelling. Due to

the natural processes which lead to the formation and modification of in-situ soil masses, soil properties

vary more or less gradually along any spatial direction. Soil properties which are measured at contiguous

spatial locations may well be expected to be closer, in magnitude, than properties measured at large spatial

distances. Depending on the complexity of the removed trend, this is also true for the set of residuals.

The more complex the trend, the less the spatial structure of residuals is preserved.

The spatial pattern which remains in the values of soil properties after the removal of a deterministic

trend can be referred to as a spatial correlation structure or, equivalently, autocorrelation, as it refers to

correlation between elements of the same set.

3.5.1 Autocorrelation and variance of residuals

An insight into the meaning of correlation and variance is warranted. Figure 33 illustrates qualitatively

the meaning of “weak” or “strong” spatial correlation and “low” and “high” variance. The horizontal and

vertical axes of the four plots of hypothetical residuals of decomposition have the same horizontal and

vertical scales. In most intuitive terms, the magnitude of spatial correlation is shown by the “roughness”

of the plot, with weak spatial correlation attesting for a “rougher” and less undulated plot than strong

spatial correlation. The magnitude of variance is reflected in the amplitude of the oscillations, with

a higher variance resulting in greater amplitude. In principle, spatial correlation and variance are not

related, i.e. the four cases are equally plausible a priori.

The decomposition procedure is to some degree arbitrary, in the sense the user should identify a

suitable trend function based on geotechnical considerations and in compliance with the requirements

of the statistical methods employed to quantify variability. As a trend is associated with exactly one set

of residuals, the latter are also (though indirectly) somewhat user-determined. The subjectivity lies in

the way in which autocorrelation is modelled, and not on the real spatial behaviour of a soil property

which is modelled as a random variable due to the user’s insufficient knowledge. Weak spatial correlation, low variance Spatial coordinate Residual values
Weak spatial correlation, high variance Spatial coordinate Residual values
Strong spatial correlation, low variance Spatial coordinate Residual values
Strong spatial correlation, high variance Spatial coordinate Residual values

Figure 33. Implications of spatial correlation and

variance. 0 5 10 15 20 25 q t [MPa] z [m] CPTU1 CPTU2
CPTU3 0 2 3 4 51

Figure 34. Superimposed plots of corrected cone tip resistance profiles from Onsøy CPTU data.

3.5.2 Anisotropy in spatial correlation

The way in which soil properties vary along different spatial directions is generally different. This is to be

expected, given the characteristics of depositional and soil modification processes. Spatial variability is,

in general, markedly anisotropic, with a larger degree of homogeneity (and, hence, a stronger correlation

structure) in the horizontal direction. This is due to mainly to the fact that most depositional processes

result in stratigraphic geometries which are usually not significantly distant from a set of horizontal

layers whose vertical thickness is much smaller than the horizontal extension. The degree of anisotropy

depends on soil type as well as in-situ conditions.

As an example, Figure 34 shows three superimposed profiles of corrected cone tip resistance q t (e.g.

Lunne et al. 1997) for Onsøy clay in Norway. Based on visual inspection, the presence of two layers (in

terms of mechanical behaviour) is evident, as well as the following: (a) the vertical autocorrelation of the

bottom layer is stronger than that of the top layer (the dispersion around a quasi-linear trend is very small

in the bottom layer, while the data scatter around any reasonably simple trend would be considerable

for the top layer); and (b) the horizontal autocorrelation is very strong for the bottom layer (the profiles

from different spatial locations are extremely similar), while this is less marked for the top layer even

though there are substantial similarities in the 3 profiles.

Anisotropy should be recognised in each phase of spatial variability estimation, starting from the

decomposition procedure (different trends should be identified for different spatial directions). It is

paramount to state clearly which spatial direction(s) the results of a spatial variability analysis refers to.

This will be shown in Section 4.

3.5.3 Distribution of residuals

The assumption of Gaussianity of residuals is strictly not necessary for the estimation of the variance

of the residuals themselves. However, it is usually an acceptable hypothesis in geotechnical spatial

variability analyses, and is convenient for the application of a number of procedures.

3.5.4 Stationarity of residuals

The main reason for performing data transformation (e.g. by decomposition) is to obtain stationary

residuals. This is desirable because virtually all classical statistics are based on the assumption of

stationarity. Stationarity will be addressed in greater detail in Section 6.

Figure 35. Scales of spatial variability modelling in geotechnical engineering (Vanmarcke 1978).

3.5.5 Fractal vs. finite-scale approaches to autocorrelation estimation

It has been stated earlier that the natural processes which result in the formation and modification of

soil masses induce a marked anisotropy in the spatial distribution of most soil properties, with the hor

izontal correlation generally being stronger. However, it is intuitive that the effects of natural processes

may occur, both at large and smaller scales. Hence, it

should be expected – for a given soil parameter of interest – that correlation structures presenting heterogeneous spatial magnitudes may be simultaneously present. Figure 35 (Vanmarcke 1978) illustrates the multiplicity of scales of investigation which are possible in geotechnical engineering: (a) soil particles; (b) laboratory specimen; (c) vertical sampling; (d) lateral distances between borings; (e) horizontal intervals at regional scale, e.g. measured along the centreline of long linear facilities. Distinct measurement techniques are generally employed for the various scales.

This suggests that the magnitude of spatial correlation also depends from the method and scale of investigation. The dependence of the correlation structure on the scale of investigation has led a number of researchers

(e.g. Agterberg 1970; Campanella et al. 1987; Fenton 1999a, 1999b; Fenton & Vanmarcke 2003; Jaksa

1995; Kulatilake & Um 2003) to address autocorrelation in soils using fractal models, which assume an

infinite correlation structure and allow to directly address the dependence of the correlation structure

from the sampling domain. Figure 36 (Fenton 1999a) provides a visual representation of a Gaussian fractal process seen at vari

ous resolutions. It may be appreciated that the profile is self-similar at every scale. For such a process,

quantitative estimates of spatial correlation would depend on the scale at which the process is observed. Assessment of the fractal nature of soil properties, though using a more geotechnical than mathematical

terminology, is common in the geotechnical engineering literature. The geotechnical engineer is well

accustomed to investigating soil properties at different spatial scales. Simonini et al. (2006), for instance,

describe the soils of the Venice lagoon as follows: “[...] the majority of soils belong to sands, silts and

silty clays with the presence of several layers of peat: the high heterogeneity is clearly evident at a larger

scale, but significant material variation may be observed even at centimetre scale.” Figure 37 provides

information regarding the large-scale vertical spatial variability of a number of soil properties from the

Malamocco site. Vanmarcke (1983) observed that it is a matter of pragmatism to recognise that, in all practical applications, there invariably exists a spatial scale below

which “micro-scale” or “macro-scale” (in relative

terms) variations are: (a) not observed or observable; and (b) of no practical interest to the situation

at hand. This observation is certainly pertinent in geotechnical engineering. As an example, Figure 38

shows the results of an investigation on the vertical spatial heterogeneity of a soil sample from the

Malamocco Venice lagoon soils investigated by Simonini et al. (2006): unit weight and void ratio vary

significantly within the sample. Lower-scale boundary is imposed by the resolution of measurement

instrumentation and the upper-scale boundary is given by the dimensions of the sample. Such pragmatic considerations justify a different approach which consists in assuming that the spatial

correlation of any soil property has a lower and upper bound, and is independent from the scale of

investigation. In this case, autocorrelation is modelled as a finite-scale stochastic process. From the point of view of numerical estimation, while the aforementioned dedicated studies have

confirmed the theoretical appropriateness of the assumption of fractal behaviour of soil properties,

practical applications (e.g. Jaksa & Fenton 2002) have shown that the vast majority of geotechnical data

Figure 36. Visual representation of a Gaussian fractal process seen at various resolutions (Fenton 1999a).

can be modelled using a finite-scale approach. Moreover, it has been suggested (e.g. Fenton 1999a;

Phoon et al. 2003c) that: (a) it is difficult to distinguish between finite-scale and fractal models over

a finite sampling domain; (b) once a reference spatial extension has been established, there may be

little difference between a properly selected finite-scale model and the real fractal model over the finite

domain.

While the use of simulation techniques could provide an effective means of establishing the relationship

between such an “effective” finite-scale model and the “true” but finite-domain fractal model, the present

state of knowledge would only allow, for practical application purposes, to refer to finite-scale models.

Hence, fractal models will not be addressed further in the present paper.

3.5.6 Finite-scale autocorrelation estimation approaches

Available finite-scale approaches for quantifying autocorrelation essentially include Bayesian and fre

quentist approaches. Baecher & Christian (2003) provided an insight into the Bayesian approach to

autocorrelation estimation. Such approach has not been widely used in the geosciences literature, and

will not be addressed further in the present paper.

Common frequentist approaches include maximum likelihood estimation and moment estimation. Sev

eral researchers (e.g. Fenton 1999a; Baecher & Christian

2003) have advocated the use of maximum

likelihood estimation over moment estimation, as the latter are more prone to bias in case of long-scale

correlation. Also, maximum likelihood allows simultaneous estimation of the spatial trend and the auto

correlation in residuals. While a number of contributions addressing maximum likelihood estimation are

available in the geotechnical literature (e.g. DeGroot & Baecher 1993; DeGroot 1996; Fenton 1999a;

Baecher & Christian 2003), most of the currently available data regarding the variability of geotechnical

parameters was obtained by moment estimation techniques.

Moment estimators use the statistical moments of a set of data (e.g. means, variances, correlation) as

estimators of the corresponding populations which are being sampled, and whose real moments are not

known. Moment estimators are conceptually and operationally simple, and have the advantage of being

non-parametric (i.e. they do not require knowledge or assumption regarding population distributions,

but only require that the moments of the distributions may be estimated). However, they may not be able

Figure 37. Results of geotechnical testing at Malamocco site in the Venice lagoon showing large-scale variability

(Simonini et al. 2006).

Figure 38. Tomographic analysis on a soil sample from the Malamocco test site in the Venice lagoon: (a) screenshot

of density in Hounsfield units; (b) bulk density in SI units and in-situ void ratio (Simonini et al. 2006).

to represent the populations sufficiently well, especially for small sample sizes, as sample moments are

always biased in practice. It is well known that the sample autocorrelation function estimated from data is “noisy” at

large

lags. The presence of noise complicates the interpretation of how the actual autocorrelation function

decays with lag distance and is clearly undesirable. However, it is rarely appreciated that the fluctuations

at large lags which reduce the quality of interpretation are expected theoretically. More precisely, the

sample autocorrelation function is a non-stationary stochastic process with mean and variance that

are functions of the lag distance. A more hopeful strategy is to exploit growing desktop computational

power and increasingly powerful simulation techniques for stochastic processes to perform bootstrapping.

Numerical results presented by Phoon & Fenton (2004) and Phoon (2006a) show that fairly accurate

mean estimates of the sample autocorrelation function can be obtained for both Gaussian and non

Gaussian processes using bootstrapping. Variance estimates are less accurate, but even crude estimates

are useful in identifying the level of noise at large lags and reducing misinterpretation of how the actual

autocorrelation function decays with lag distance.

3.5.6.1 Autocorrelation function

The autocorrelation function of a stochastic process (which must be at least weakly stationary) describes

the variation of the strength of spatial correlation as a function of the spatial separation distance between

two spatial locations at which data are available.

As stated previously, it is not possible to calculate the real autocorrelation function of a stochastic

process (and, thus, to investigate its true spatial correlation structure) due to the fact that data sets are,

in practice, always limited in size. Hence, it is necessary to refer to the sample autocorrelation function,

i.e. to an approximation of the real autocorrelation function, calculated from an available set of data

which is deemed representative of the stochastic process.

In the discrete case, the sample autocorrelation sample of a set of data is given by:

in which $\bar{x}$ is the mean of the data set. The function should be estimated for separation distances

$| \tau_j | = j\Delta z$ corresponding to $j = 1, 2, \dots, n/4$, as suggested by Box & Jenkins (1970), where n is the

number of data points and Δz is the (constant) sampling interval.

It should be noted that absolute values have been used to denote separation distance. This is due to the

fact that the autocorrelation function is symmetric about zero separation distance (essentially represents

the fact that autocorrelation can be investigated in both directions from a given spatial location). However,

due to the aforementioned symmetry, only the positive separation distance domain of autocorrelation

functions is usually represented graphically.

Unless specific techniques such as the Galerkin polynomial trend fitting method are used, decom

position results in a zero-mean set of residuals. In the case of zero-mean sets, Eq. (27) can be

rewritten as:

The autocorrelation at zero separation distance is equal to 1.0, which is its maximum value; empirically,

for most geotechnical data, autocorrelation tends to zero with increasing separation distance, and can

assume negative values.

Figure 39 provides a visual comparison between two sample autocorrelation functions: ACF1, calculated

from a data set with a weaker spatial correlation structure, decreases more rapidly to zero than

ACF2, representing a data set with stronger correlation structure. A data set with no spatial correlation

(i.e. white noise) has unit autocorrelation at zero separation distance and zero autocorrelation otherwise.

Fenton (1999a) showed that the sample autocorrelation function is useful in the estimation of spatial

correlation if such correlation is considerably smaller than the extension of the sampling domain. If, on

the contrary, the process has long-scale dependence, the sample autocorrelation function is unable to

detect this, and continues to illustrate short-scale behaviour.

The autocorrelation function has been widely used for investigating spatial variability in the context

of geotechnical engineering (e.g. Akkaya & Vanmarcke 2003; Baecher 1982, 1986, Baecher & Christian

Figure 39. Examples of autocorrelation functions with weaker (ACF1) and stronger (ACF2) spatial correlation.
Table 17. Selected theoretical autocorrelation models as a function of separation distance τ . Autocorrelation model
Formula Single exponential (SNX) $R(\tau) = \exp(-k_{SNX} |\tau|)$
Cosine exponential (CSX) $R(\tau) = \exp(-k_{CSX} |\tau|) \cos(k_{CSX} \tau)$
Second-order Markov (SMK) $R(\tau) = (1 + k_{SMK} |\tau|) \exp(-k_{SMK} |\tau|)$
Squared exponential (SQX) $R(\tau) = \exp[-(k_{SQX} \tau)^2]$

2003; Cafaro et al. 2000; Cafaro & Cherubini 2002; Campanella et al. 1987; Christian et al. 1994;

DeGroot & Baecher 1993; DeGroot 1996; Fenton 1999b; Fenton & Vanmarcke 2003; Jaksa 1995; Jaksa

et al. 1997, 2000; Phoon & Fenton (2004); Phoon & Kulhawy 1996, 1999a, 1999b; Phoon et al. 2003c,

2004; Uzielli 2004; Uzielli et al. 2005a, 2005b; Vanmarcke

1983; White 1993; Wickremesinghe &

Campanella 1993; Wu & El-Jandali 1985; Wu 2003). To identify how a given soil properties varies spatially, one needs to fit one or more autocorrelation

models to the sample autocorrelation function calculated by (28). An autocorrelation model quantifies

autocorrelation as a function of separation distance, and is described by a function and a characteristic

model parameter. Various autocorrelation models have been employed in the geotechnical literature (e.g.

Spry et al. 1988; DeGroot & Baecher 1993; Jaksa 1995; Lacasse & Nadim 1996; Fenton 1999b; Phoon

et al. 2003c; Uzielli et al. 2005a, 2005b). The equations relating the autocorrelation function, $R(\tau)$, and

separation distance (τ) for four models are given in Table 17. According to Spry et al. (1988), no autocorrelation model is univocally preferable over others on the

basis of physical motivations. Figure 40 (Uzielli et al. 2005a) shows, for instance, the distribution of

best-fit autocorrelation models for soil units which were found to be physically homogeneous in terms

of stress-normalised cone tip resistance. Each point on the CPT-based soil behaviour classification

chart by Robertson (1990) represents the mean values of normalised friction ratio F/R (Wroth 1984) and

normalised cone tip resistance q/c_1N (Robertson & Wride 1998) in a physically homogeneous soil unit. It

may be observed that no relation exists between best-fit autocorrelation models and soil type (cohesive

soils in zones 2 and 3; intermediate behaviour soils in zone 4 and cohesionless soils in zones 5 and 6),

as well as data points representative of profiles from physically homogeneous soil units for which weak

stationarity could not be investigated in a rigorous way

using the MBSR method (see 86.1.2) because

of insufficiently reliable estimates of the scale of fluctuation (NAPP) or because they were classified as

non-stationary (NST). Future research could contribute to a clarification of the issue. As there does not appear to be a reliable physical correspondence between soil type and the type of

correlation structure, a purely numerical approach is justified. The choice of the correlation structure

for a given data set can be based, for instance, on the comparative assessment of goodness-of-fit of

one or more theoretical autocorrelation models to the empirical sample autocorrelation function of the

data set itself. The procedure requires the optimisation of the characteristic model parameter for each

autocorrelation model (e.g. by least-squares regression or other optimisation procedures) and the sub

sequent calculation of the goodness-of-fit parameter. The autocorrelation model yielding the maximum

determination coefficient could be selected as best-fit model.

Figure 40. Distribution of best-fit autocorrelation models for stress-normalised cone tip resistance in physically

homogeneous soil units (Uzielli et al. 2005a).

Figure 41. Fit of single exponential (SNX) model curves with different characteristic model parameters to an

empirically calculated sample autocorrelation function.

Figure 41 illustrates the meaning of optimisation for a specific autocorrelation model. Two curves

from the single exponential model (one with $k_{SNX} = 2$ and $k_{SNX} = 4$) are fitted to a sample autocorrelation

function. It can be observed that the curve with $k_{SNX} = 2$ provides a significantly better fit.

Figure 42 shows the importance of identifying the most suitable autocorrelation model. In the

figure, the optimised single exponential and squared exponential models are fitted to a sample

autocorrelation function. It may be seen that the single exponential model approximates the correlation

structure of the data set better than the squared exponential.

Figure 43 illustrates a real-case application of the autocorrelation model fitting explained above for

data from a CPTU sounding performed in Brindisi, Italy (Cherubini et al. 2006). The sample autocorre

lation function is shown along with four theoretical autocorrelation models which characteristic model

parameters have been optimised. The single exponential model with $k_{SNX} = 1.23$ provides the best fit.

With the aim of setting stricter criteria on the choice of the correlation structure of the residuals, Spry

et al. (1988) opined that the autocorrelation structure should be identified based only to the initial part of

Figure 42. Curve fit of single exponential (SNX) and squared exponential (SQX) models to an empirically calculated

sample autocorrelation function.

Figure 43. Sample autocorrelation function and autocorrelation model fitting for a soil from the Brindisi area

(Cherubini et al. 2006).

the sample autocorrelation function, i.e. by fitting empirical autocorrelation models to the autocorrelation

coefficients exceeding Bartlett's limits, which can be approximated for practical purposes by:

This restrictive condition is motivated by the fact that the estimated autocorrelation coefficients become

less reliable with increasing separation distance, and are deemed not significantly different from zero

inside the range $\pm 1 B$ (e.g. Priestley 1981; Fenton 1999a). With the aim of establishing yet more restrictive criteria for the acceptance of the correlation structure,

Uzielli (2004) suggested that a minimum R^2 value be set for the autocorrelation model fit to the sample

autocorrelation function. Moreover, to ensure that the fit is significant, Uzielli (2004) opined that at

least 4 autocorrelation coefficients should be used to fit the autocorrelation model and to calculate the

fit parameter R^2 . Table 18. Selected theoretical semivariogram models as a function of separation distance τ . Semivariogram model Formula Spherical (SPH) $V(\tau) = C \left(\frac{3}{2} \left[\frac{\tau}{a} - \frac{\tau^3}{a^3} \right] \right) + C_0$ for $|\tau| \leq a$ SPH $V(\tau) = C + C_0$ for $|\tau| > a$ SPH Exponential (EXP) $V(\tau) = C \left[1 - \exp \left(- \frac{|\tau|}{a} \right) \right] + C_0$ Gaussian (GAU) $V(\tau) = C \left[1 - \exp \left(- \frac{|\tau|^2}{a^2} \right) \right] + C_0$

3.5.6.2 Semivariogram

The semivariogram provides essentially the same information as the autocorrelation function. However,

it has the advantage of being an unbiased estimator as it does not depend on the mean of the data set. As

for the autocorrelation function, the exact shape of the semivariogram cannot be known from a sample

of limited size. Hence, it is necessary to estimate the sample semivariogram of a data set of n elements:

(for $j = 0, 1, \dots, n - 1$). The sample semivariogram may be used to investigate the spatial correlation

structure of a specific data set.

The semivariogram requires a less restrictive statistical assumption regarding stationarity than the

autocovariance function, as it does not require stationarity of the mean (in other words, trends in data

can be accepted). However, it has often been recommended (e.g. Journel and Huijbregts 1978) that data

should be detrended. Theoretical semivariogram models are available; a number of these is illustrated in

Table 18.

The parameter C_0 is referred to as the nugget, and represents small-scale variability; the parameter a

is termed range, and describes the maximum distance at which correlation between observations may

be assumed; $C + C_0$ is the sill.

All of the above models obey the condition $V(0) = 0$. However, C_0 generally assumes non-zero values,

and it appears that the semivariogram has a non-zero intercept. This is representative of the nugget effect,

and attests for the existence of a lower-bound spatial distance in which the random process of interest is

so erratic that the semivariogram goes from zero to C_0 in a distance less than the sampling interval. The

nugget effect (addressed in greater detail in Jaksa 1995) is due essentially to small-scale variability as

well as measurement error. The two components are indistinguishable in practice. A discussion on small

scale variability is provided in §3.7. Figure 44 provides a comparative plot of the spherical, exponential

and gaussian theoretical semivariogram models, with range equal to unity ($a_{SPH} = a_{EXP} = a_{GAU} = 1$) and

$C_0 = 1$.

Investigation of the correlation structure by means of the semivariogram is conceptually analogous to

the procedure based on the sample autocorrelation function. Optimisation for one or more semivariogram

models is followed by the selection of the best-fit semivariogram model. Journel and Huijbregts (1978)

suggested that automatic fitting of models to experimental semivariograms (e.g. by least-squares) should

be avoided because each point of an empirical semivariogram is calculated using a different number of

data points (as a function of distance); hence, the estimation error is not uniform. Hence, a weighted fit

(with weighting function based on critical appraisal of data and on experience) is preferable. In the case

of the autocorrelation function, Bartlett's limits can be adopted as a threshold for determining which

autocorrelation coefficients are sufficiently reliable; no equivalent parameters are available for experi

mental semivariograms. Hence, the use of the latter is somewhat more uncertain in comparison with

the former. Figure 45 (Jaksa 1995) provides visual examples of: (a) excellent fit; and (b) poor fit of the

spherical model to sample semivariograms.

The semivariogram has frequently been used in the geotechnical literature (e.g. Azzouz et al. 1987;

Azzouz & Bacconnet 1991; Baecher 1984; Brooker et al. 1995; Chiasson et al. 1995; DeGroot 1996;

Elkateb et al. 2003a, 2003b; Fenton 1999a; Hegazy et al. 1996; Jaksa 1995; Jaksa et al. 1993; Krige

1951; Kulatilake & Miller 1987; Kulatilake & Ghosh 1988; Kulatilake & Um 2003; Matheron 1963;

Nobre & Sykes 1992; Nowatzky et al. 1989; O'Neill & Yoon 2003; Oullet et al. 1987; Soulié 1983;

Soulié et al. 1990; White 1993).

Figure 44. Plot of semivariogram models for range equal to unity and $C_0 = 1$. (a) (b)

Figure 45. Example fits of sample semivariograms to spherical semivariogram model: (a) excellent fit; and (b) poor

fit (Jaksa 1995).

3.6 Quality of data for autocorrelation analysis

In the second-moment perspective, sample numerosity and outlier exclusion were sufficient criteria

for assessing the quality of a data set (statistical estimation error is reduced). In addressing the spatial

variability of data and autocorrelation, additional criteria are required. For instance, the techniques

used to model soil variability are greatly simplified if the measurement interval in data sets is constant.

Alonso & Krizek (1975) opined that a good definition of soil variability requires that continuous or

quasi-continuous records of soil properties be used “to show both the random character of the soil mass

and the correlation of properties from point to point”. For non-uniformly spaced data, the sample autocorrelation function can still be used, but should, in

strict terms, require dividing separation distances into bands, and then taking the averages of autocor

relation within those bands (Baecher & Christian 2003). In practice, for testing methods with frequent

recordings such as the CPT, measurement depths can be rounded to regular interval series and assuming

strong property autocorrelation within the reference interval without introducing significant errors. Jaksa

(1995) referred to this procedure as rationalisation. For high-interval tests such as the SPT it is harder

to perform rationalisation as the hypothesis of strong correlation over significantly longer measurement

intervals is much less acceptable.

3.7 Small-scale variability

It has been stated previously that a scatter due to

small-scale but real variations in a measured soil

property is at times mistakenly attributed to measurement error. In practice, it is extremely difficult to

separate such sources of uncertainty. It should be noted, however, that laboratory and in-situ geotechnical measurements are based on the

generation of some type of failure and plastic deformation of a spatial volume of soil. Thus, measurements

Figure 46. Example application of Baecher's procedure for the estimation of random measurement error (Jaksa

et al. 1997).

are not point measurements, but are representative parameters of the extent of the volume in which such

failure occurs. The magnitude of the spatial averaging effect is greater than the magnitude of small-scale

variation (Vanmarcke 1983). Hence, in the context of practical variability analyses, the attribution of

small-scale variations to soil variability or measurement error is of limited relevance.

3.7.1 Baecher's procedure

Baecher (1982) proposed an indirect method for the quantification of small-scale variability. The method

consists in the back-interpolation of the sample autocorrelation function (with the exclusion of the origin)

to zero separation distance of the autocorrelation plot. A zero-lag autocorrelation coefficient r_0 . Figure

46 (Jaksa et al. 1997) illustrates an application of the method.

3.7.2 Semivariogram nugget

As described in §3.5.6.2, the semivariogram nugget C_0 provides a quantitative measure of the nugget

effect, which is related to small-scale variability.

3.7.3 Autocorrelation nugget

Jaksa et al. (1997) defined the autocorrelation function nugget R_0 as the difference between unity and

the autocorrelation intercept from Baecher's procedure r_0 :

The autocorrelation function (ACF) nugget provides a measure of the magnitude of small-scale variability,

i.e. the residual autocorrelation at separation distances smaller than measurement spacing. It is

conceptually analogous to the semivariogram nugget, but is easier to interpret numerically as it ranges

from 0 to 1.

Jaksa et al. (1997) investigated the effect of trend removal and sample spacing on the autocorrelation

nugget of profiles of cone tip resistance in the highly homogeneous Keswick clay. Table 19 illustrates

the dependence of weak stationarity, fit of trend (given by the coefficient of determination R^2) and

ACF nugget on the degree of the polynomial trend removed from the source profiles. The ACF nugget

increases monotonically with increasing complexity of the polynomial trend function.

Table 20 (Jaksa et al. 1997) illustrates the dependence of the ACF nugget on variations in sample

spacing of profiles achieved by sampling source data at differing intervals. While the lower bounds in

the range of ACF nuggets do vary, the upper bounds increase significantly with sample spacing.

4 RANDOM FIELD MODELLING OF SPATIAL VARIABILITY

The modelling of spatial correlation using autocorrelation function or semivariogram does not allow full

representation of the effects of soil variability on the performance and reliability of spatially random Table 19. Dependence of weak stationarity, fit of trend and ACF

nugget on removed polynomial trend for cone tip resistance data in the vertical direction (adapted from Jaksa et al. 1997). Degree of polyn. trend Weak stationarity Degree of trend fit (R^2) ACF nugget [%] none No - 5 1 No 0.08 6 2 Yes 0.72 12 3 Yes 0.73 12 4 Yes 0.77 13 5 Yes 0.82 14 6 Yes 0.85 22 Table 20. Dependence of ACF nugget on sample spacing for cone tip resistance data in the vertical direction (adapted from Jaksa et al. 1997). Sample spacing [mm] Number of data sets ACF nugget [%] 5 1 10 10 2 10-11 20 4 5-7 50 5 3-24 100 5 18-60 200 5 3-62

geotechnical systems. Important effects such as the spatial averaging effect and the existence of a critical

spatial correlation require additional operations and parameters. Spatial variability can be expressed concisely by means of random field theory. In most general

terms, a random field is essentially a random (or stochastic) process consisting of an indexed (i.e.

ordered according to one or more reference dimension) set of random variables (Vanmarcke 1983). In an applicative geotechnical perspective, random field theory is important for two main reasons: first,

it provides a vehicle for incorporating spatial variation in engineering and reliability models (e.g. Uzielli

et al. 2005a); second, through geostatistics, it provides powerful statistical results which can be used to

draw inferences from field observations and plan spatial sampling strategies (e.g. Jaksa et al. 2005). In the present chapter, the spatial averaging effect and variance reduction due to spatial averaging are

addressed. The scale of fluctuation and the coefficient of variation of inherent variability are introduced

(among others available in the literature) as concise descriptors of a random field in the second-moment

sense. A review of calculation methods for such parameters is provided, as well as selected literature data. It is anticipated here that variance reduction alone cannot explain the behaviour and reliability of

complex geotechnical systems. As will be discussed in Section 7, actual failure mechanisms in 2D or 3D

geotechnical problems will involve failure surfaces which do not correspond to the pre-determined ones

assumed by traditional approaches (e.g. circular surfaces in some slope stability methods). At present,

simulation seems the only viable brute-force solution. Details and examples are discussed in Section 7.

4.1 The issue of scale in random field modelling

It has been stated previously that the finite-scale correlation approach is preferable, at present, for

practical evaluation of the spatial variability of soil properties. With reference to finite-scale approaches,

Vanmarcke (1983) stated that "The question of scale is of paramount importance in practical applications

of random field theory. A phenomenon that appears deterministic on the micro-scale [...] may at a larger

scale exhibit highly variable properties that call for probabilistic description. On an even larger scale,

these same structures may be embedded in objects amenable to characterisation in terms of average [...]

properties. [...] In other words, there is usually a lower and an upper bound on the range of dimensions

within which a random field model has practical value." It is of interest to investigate the physical reasons which support the hypothesis of the existence of

upper and lower bounds to spatial correlation in the context of random field modelling. An upper bound to

spatial correlation is given by the always finite extension of any sampling domain. As has been discussed

in greater detail in §3.7, it is virtually impossible to distinguish small-scale variation from measurement

Figure 47. Representation of spatial averaging of random processes measured at discrete, constant spatial interval:

(a) non-averaged random process; (b) same random process

averaged over an interval T_1 ; (c) same random process

averaged over an interval $T_2 > T_1$. Note: x-axis and y-axis scales are the same in all diagrams.

error. Hence, random field models need only provide information about random variables on a scale

which is sufficient to represent behaviour under at least some amount of spatial averaging.

4.2 Variance reduction due to spatial averaging

An implicit manifestation of spatial correlation which is commonly encountered in geotechnical engin

neering practice is that the representative value of any soil property depends on the volume concerned in

the problem to be solved. With reference to a given soil unit and to a specific problem, the geotechnical

engineer is trained to define the design values of relevant parameters on the basis of the magnitude of

the volume of soil governing the design.

Any laboratory or in-situ geotechnical measurement includes some degree of spatial averaging in

practice, as tests are never indicative of point properties, but, rather, are used to represent volumes of soil.

Sets of measurements are always discrete as there is always some spatial separation between two

measurement locations. The local average $\bar{T}$ of a discrete random process over a spatial interval T

is defined by

in which n is the number of measurements in T and Δz is the (constant) discrete measurement interval.

Figure 47 provides an example of the effect of spatial averaging: in (a), the non-averaged process (i.e. a

generic set of residuals) is shown; in (b) and (c), the same process is shown following spatial averaging

for a spatial interval T_1 and T_2 , respectively, with $T_2 > T_1$.

The spatial averaging effect results in a reduction of the effect of spatial variability on the computed

performance because the variability is averaged over a volume, and only the averaged contribution to the

uncertainty is of interest as it is representative of the “real” physical behaviour (e.g. Lacasse & Nadim

1996).

Figure 48 provides an example of the variance reduction effect due to spatial averaging of a generic

parameter in the two-dimensional case. The resulting 3 histograms show that the mean varies little due to

spatial averaging (depending on the distribution of the process), but the variance decreases significantly

from the “1 × 1 block” (non-averaged) case to the “5 × 5 block” and to the “10 × 10 block” case.

Figure 48. Effect of spatial averaging on variance reduction (El-Ramly et al. 2002). The reduction in variance due to spatial averaging effect can be represented by the variance function

(Vanmarcke 1983), which expresses the ratio of the variance of a spatially averaged random process to

that of the same (non-averaged) process. In other words, it provides a measure of the reduction of the

point variance under spatial averaging as a function of the spatial averaging distance T :

in which s_T is the variance of the trend function and s is the total variance of the data set. The

variance reduction functions of theoretical autocorrelation models can also be expressed concisely by

mathematical expressions (e.g. Vanmarcke 1983). However, as for the autocorrelation function and

semivariogram, the limited size of data sets makes it necessary to estimate the variance reduction based

on samples which are always limited in size and are composed of measurements obtained at discrete

spatial intervals. Given a series of n equally spaced observations over a sampling domain of size T , the

discrete approximation of the i -th value of the sample variance reduction function is

for $i = 1, 2, \dots, n$, where

Figure 49. Example of a discrete variance reduction function.

for $j = 1, 2, \dots, n - i + 1$. Figure 49 shows a typical variance reduction function calculated in the discrete

case using Eq. (34) and (35). From the figure it is inferred, for instance, that the variance of the random

process averaged over 6 m is equal to the variance of the non-averaged process multiplied by 0.28.

The condition of at least weak stationarity is necessary because the variance reduction function

expresses the reduction in the variance of an averaged process as a function of the averaging interval

only, and not on the spatial position of a subset of the random process itself (i.e. whether the spatial

averaging occurs in the initial or terminal section of a profile).

The results obtained for the one-dimensional case may be extended to the two and three-dimensional

cases by assuming a separable, independent correlation structure for each spatial dimension. Hence, the

total variance reduction can be calculated as the product of the variance reductions in the individual

dimensions. The reader is referred to Vanmarcke (1983) and Elkateb et al. (2003a) for further insights

into multi-dimensional variance reduction.

4.3 Scale of fluctuation

Vanmarcke (1983) defined the scale of fluctuation as the proportionality constant in the limiting expres

sion of the variance function, i.e. as the value assumed by the product of the variance function and the

spatial averaging extension when the latter tends to infinity (or, in practice, becomes large):

In finite-scale models, the scale of fluctuation, δ , is a concise indicator of the spatial extension of the

correlation structure. Within separation distances smaller than the scale of fluctuation, the deviations

from the trend function are expected to show relatively strong correlation. When the separation distance

between two sample points exceeds the scale of fluctuation, it can be assumed that little correlation exists

between the fluctuations in the measurements.

The scale of fluctuation is an extremely important parameter for the representation of a random field.

Actually, a random field is described (in the second-moment sense) by its mean, standard deviation

and scale of fluctuation, as well as by a functional form for the autocorrelation function. The scale of

fluctuation is also useful to approximate the variance reduction function, thereby allowing to avoid use

of Eq. (34) and (35) (see §4.6).

The scale of fluctuation of a random process does not always exist. Vanmarcke (1983) showed that

the condition for the existence of the scale of fluctuation is (in simple, qualitative terms) is that the

autocorrelation function converges to zero with increasing separation distance.

Other concise descriptors of spatial correlation have been employed in the geotechnical literature.

These include autocorrelation distance and effective semivariogram range. The interested reader is

referred to Jaksa (1995), Lacasse & Nadim (1996) and Elkateb et al. (2003a). As each represents distinct

characteristics of spatial correlation structure, the respective numerical values are different for a given Table 21. Scale of fluctuation for selected autocorrelation models. Model Scale of fluctuation Single exponential (SNX) $\delta = 2/k$ SNX Cosine exponential (CSX) $\delta = 1/k$ CSX Second-order Markov (SMK) $\delta = 4/k$ SMK Squared exponential (SQX) $\delta = \sqrt{\pi/k}$ SQX

Figure 50. Autocorrelation models for scale of fluctuation equal to 1.00 m.

autocorrelation function (or semivariogram); hence, care should be taken as to avoid inadvertently using

one in place of the other. A variety of techniques for the estimation of the scale of fluctuation are available in the geotechnical

literature. A number of these are presented briefly in the following.

4.3.1 Fitting of autocorrelation models (AMF)

The scale of fluctuation can be calculated from the fitting of theoretical autocorrelation models

to empirically calculated sample autocorrelation functions. When the best-fit autocorrelation model

has identified by regression, the characteristic model parameter can be obtained as explained in

§3.5.6.1. The scale of fluctuation may then be evaluated by applying the appropriate analytical

relationship with the characteristic model parameter of the best-fit model among those shown in

Table 21. Figure 50 provides a comparative plot of the SNX, CSX, SMK and SQX autocorrelation models

for a unit scale of fluctuation, i.e. for $k_{SNX} = 2$; $k_{CSX} = 1$; $k_{SMK} = 4$; and $k_{SQX} = \sqrt{\pi}$. While the models

appear to be quite similar, the importance of the different behaviours at low separation distances (i.e.

where the autocorrelation coefficients are estimated most reliably) should not be downplayed. With

reference to Figure 50, for instance, at a separation distance $\tau = 0.25$ m, the single exponential and

square exponential models would provide, respectively, autocorrelation coefficients of 0.61 and 0.82,

which indicate considerably different magnitude of spatial correlation! Hence, the identification of

the most suitable correlation structure should be performed with utmost care and as rigorously as

possible. Figure 51a (Phoon 2006b) shows a detrended normalized cone tip resistance (q_{c1N}) profile spanning

between 14.125 m and 27.975 m in depth. The sampling interval is 2.5 cm. The empirical autocorrelation

function can be fitted to a single exponential function or a square exponential function with a scale of

fluctuation = 0.98 m (Figure 51b). The simulated profiles are shown in Figure 52. The low frequency fluctuations are quite accurate for

both models. The high frequency fluctuations are over-represented for the single exponential model and

under-represented for the square exponential model. The reason is that the rate of decay is too fast for

the single exponential model and too slow for the square exponential model. (a) (b) 14 16 18 20 22 24 26 28 0

-100

-50

50

100 Depth (m)

D
 e t
 r e
 n d
 e d
 n
 o r
 m
 a l
 i z
 e d
 c
 o n
 e
 t i p
 r e
 s i s
 t a
 n c

e 0 0.5 1 1.5 2 2.5 3 -0.2 0 0.2 0.4 0.6 0.8 τ R (τ)
 Single exponential fit Square exponential fit Empirical
 autocorrelation

Figure 51. (a) Detrended normalized cone tip resistance
 profile and (b) Empirical autocorrelation function fitted
 with two theoretical models (Phoon 2006b). (a) (b) 14 16
 18 20 22 24 26 28

-50 0 50

100 Depth (m)

D

e t

r e

n d

e d

n

o r

m

a l

i z

e d

c

o n

e

t i p

r e

s i s

t a

n c

e Measurement Simulation 1 Simulation 2 14 16 18 20 22 24
26 28 -100 -50 0 50 100 Depth (m) D e t r e n d e d n o r m
a l i z e d c o n e t i p r e s i s t a n c e Measurement
Simulation 1 Simulation 2

Figure 52. Simulated profiles from (a) Single exponential
model and (b) Square exponential model (Phoon 2006b).

4.3.2 Integration of sample autocorrelation function (SAI)

Vanmarcke (1983) showed that the scale of fluctuation of a random process, if it exists, is also given by

the area under the autocorrelation function or, since the latter is symmetric about the origin, to twice the

area below the positive-separation distance semi-function.

In the discrete case, this can be approximated by Eq. (37):

where $\hat{R}$ is the estimated sample autocorrelation function (defined in §3.5.6.1) and $\Delta\tau$ is the constant

separation distance between two consecutive measurements. As the scale of fluctuation is not well defined

for autocorrelation functions which do not converge to zero relatively rapidly, m is usually taken to be

the index of the separation distance where the sample correlation function first becomes negative (Jaksa

1995). An example application of the SAI technique to a detrended q_{c1N} profile is shown in Figure 53.

4.3.3 Bartlett limits method (BLM)

Jaksa (1995) observed that the scale of fluctuation of cone tip resistance in clays was very well approxi

mated by Bartlett's distance r_B , i.e. the separation distance corresponding to the first intersection of the

sample autocorrelation function with Bartlett's limits [see Eq. (29)]. An advantage of this method is that

the estimated scale of fluctuation is not affected by measurement interval (Jaksa 1995).

Figure 53. Application of the SAI technique for the estimation of the scale of fluctuation (Uzielli 2004).

Figure 54. Example of application of the BLM technique for the estimation of the scale of fluctuation (Jaksa 2006). Uzielli (2004) asserted that only scales of fluctuation obtained from sample autocorrelation functions

definitively oscillating inside Bartlett's limits should be accepted. An application of the BLM technique

to cone penetration resistance data (Jaksa 2006) is shown in Figure 54.

4.3.4 Simplified method (VXP)

Vanmarcke (1977) proposed the following approximate relationship for evaluating the scale of

fluctuation:

where

is the average distance between the intersections of the fluctuating component and the trend of a given

profile (see Figure 55).

4.3.5 Fluctuation function method (VRF)

Based on the definition by Vanmarcke (1983), the scale of fluctuation can be estimated in practice as

the limit value of the fluctuation function, i.e. the product of the variance function and the separation

distance when the latter becomes "large". Such procedure has been employed in several studies (e.g.

Wickremesinghe & Campanella 1993; Cafaro & Cherubini 2002; Uzielli 2004). A unique criterion to identify "large separation distance" is not available. Wickremesinghe & Cam

panella (1993) suggested that the separation distance corresponding to the point of inflection of the

variance reduction function may be used; Uzielli (2004) adopted the separation distance corresponding

to a variance reduction coefficient of 0.15.

Figure 55. Mean-crossings approximation for the estimation of the scale of fluctuation (Vanmarcke 1977).

Figure 56. Application of the VRF technique for the estimation of the scale of fluctuation: (a) variance reduction

function; (b) fluctuation function (Uzielli 2004).

Fenton (1999a) showed that the sample variance function becomes unbiased only if the averaging

region grows large and the correlation function decreases rapidly within the averaging region itself.

Like the sample correlation function, the sample variance function is really only a useful estimate of

second-moment behaviour for short-scale processes, i.e. processes whose spatial correlation distance is

small in comparison with their spatial extension.

An example application of the VRF technique to a detrended cone penetration resistance profile

(Uzielli 2004) is shown in Figure 56. In the example, the variance reduction function attains a value of

0.15 at 68 lags (i.e. data points) from the origin, equivalent to a separation distance of 1.34 m because

the measurement interval is 2 cm. Hence, the scale of fluctuation is taken as the value of the fluctuation

function at $\tau = 1.34$ m.

4.3.6 Estimation from semivariogram range (SVR)

Similarly to the AMF method for the autocorrelation functions, the scale of fluctuation can be calculated

from the fitting of theoretical semivariogram models to empirically calculated sample semivariograms.

When the best-fit semivariogram model has been identified by regression, the characteristic model

parameter can be obtained, and the scale of fluctuation may then be evaluated by applying the appropriate

analytical relationship with the characteristic model parameter of the best-fit model among those shown

in Table 22.

4.4 Factors influencing the scale of fluctuation

The scale of fluctuation is not an inherent property of a soil parameter. Estimated values of the scale of

fluctuation are closely bound to the estimation methodology, as they depend at least on: (a) the spatial Table 22. Relation between scale of fluctuation and characteristic semivariogram model parameters (Elkateb et al. 2003a). Model Scale of fluctuation SPH $\delta = 0.55a$ SPH EXP $\delta = 2a$ EXP /3 GAU $\delta = a$ GAU

Figure 57. Profiles from cone penetration, flat dilatometer, seismic cone and cross-hole testing from the Malamocco

site in the Venice lagoon, showing the effect of the difference in measurement interval (Simonini et al. 2006).

direction [e.g. horizontal, vertical]; (b) the measurement interval in the source data; (c) the type of trend

which is removed during decomposition; (d) the method of estimation of the scale of fluctuation from

residuals [e.g. fluctuation function method vs. autocorrelation model fitting]; and (e) modelling options

from the specific estimation method [e.g. choice of best-fit autocorrelation model]. Point (a) refers to the inherent anisotropy of soils and in-situ soil masses. From a geological and

geomorphological perspective, it is intuitive that soil formation and modification processes, as well as

factors contributing to the definition of the in-situ state (e.g. stress) would result in a greater heterogeneity

of soil properties in the vertical direction and, hence, in a weaker spatial correlation. As will be shown in

§4.5, the scale of fluctuation of a given soil property in the vertical direction is generally much smaller

than in the horizontal direction. Regarding point (b), research has shown that the scale of fluctuation depends on the measurement

interval. More specifically, it tends to increase with sample spacing. In geotechnical site characterisation,

measurement spacing varies significantly among testing methods. Figure 57 (Simonini et al. 2006)

illustrates the difference in resolution of cone penetration tests, flat dilatometer tests, seismic cone and

cross-hole tests from the Malamocco site in the Venice lagoon. Cafaro & Cherubini (2002) evaluated the scale of fluctuation of data sets of cone tip resistance in

two clay layers from a Southern Italian site using the VRF method. The sets were sampled at different

Table 23. Vertical scales of fluctuation of cone tip resistance calculated using quadratic detrending and the VRF

method for different measurement intervals and for soundings G1-G15 in two clay layers in Southern Italy (Cafaro &

Cherubini 2002).

Spacing [m] G1 [m] G3 [m] G6 [m] G7 [m] G15 [m] Mean [m]

Upper clay

0.20 0.20 0.40 0.21 0.40 0.44 0.33

0.40 0.20 0.34 0.26 0.44 0.51 0.35

0.60 0.19 0.35 0.24 0.52 0.47 0.36

Lower clay

0.20 0.54 0.29 0.72 0.27 0.19 0.40

0.40 0.64 0.39 0.67 0.28 0.23 0.44

0.60 0.41 0.37 0.96 0.28 0.53 0.51

Table 24. Influence of trend removal on the scale of fluctuation of cone tip resistance of an Italian clay calculated

using the VRF method (Cafaro & Cherubini 2002). Scale of fluctuation [m] for Scale of fluctuation [m] for

Sounding linear trend removal quadratic trend removal

G1 0.42 0.20

G3 0.40 0.40

G6 0.60 0.21

G7 0.96 0.40

G15 0.47 0.44

measurement intervals: 0.2, 0.4 and 0.6 m. The results, reported in Table 23, indicate that the scale of

fluctuation tends to increase with measurement interval.

Trend removal generally results in a decrease in the estimated scale of fluctuation, because, as seen

previously, the trend accounts for some spatial correlation. Moreover, for a given estimation methodology,

the scale of fluctuation decreases with increasing complexity of the trend. Table 24, which summarises

the results of an investigation by Cafaro & Cherubini (2002) on cone resistance values measured at

0.20m intervals, provides an example of such phenomenon.

Regarding the influence of the estimation method, Uzielli (2004) compared the estimates of the scale

of fluctuation of profiles of normalised cone tip resistance using the SAI, VXP, BLM, VRF and AMF

estimation methods described in §4.3. The results are plotted in Figure 58. Each stack of values refers to

a weakly stationary, physically homogeneous soil unit. Measurement intervals for the different profiles

ranged from 0.002 to 0.10 m. By examination of Figure 58, it may be stated that none of the models

rank consistently above or below the others; however, in some cases the scatter among estimates from

different methods is significant.

Jaksa (1995) investigated the relationship between Bartlett's distance and the scale of fluctuation

(obtained by autocorrelation model fitting) for Keswick clay. The strong regression fit, shown in Figure

59, attests for the good agreement between the methods in the specific case.

Based on the above observations, it may be stated that numerical values of the scale of fluctuation

should thus be viewed with extreme caution. Strictly speaking, they should only be used for inferential

purposes if the measurement interval, the removed trend type and the estimation methodology are

explicitly reported along with the numerical estimate.

Despite the marked dependence on the estimation procedure, research has shown (e.g. Phoon &

Kulhawy 1999; Uzielli et al. 2005a, 2005b) that, if the vertical scale of fluctuation of different soil

types (e.g. cohesive vs. cohesionless) and the same conditions are maintained with regards to (a)-(e),

a dependency on soil type is generally observed.

Figure 60 reports some of the results of an investigation by Uzielli et al. (2005a) on random field

characterisation of stress-normalised cone penetration parameters, namely the normalised, dimensionless

cone penetration resistance q_{c1N} and the normalised friction ratio F_R .

Scales of fluctuation in the vertical direction were estimated by fitting autocorrelation models to

linearly detrended data from highly physically homogeneous soil units from five regional sites worldwide.

These were identified using a procedure described in Section 5. In the plots, each point represents one soil

unit. As normalised cone tip resistance is related to soil

type (with resistance decreasing with increasing

cohesive behaviour), the plots can be useful in investigating the existence of a soil-type effect on vertical

Figure 58. Estimates of the vertical scale of fluctuation for q_{c1} using BLM, VXP, VRF, SAI and AMF (Uzielli 2004).

Figure 59. Empirical relationship between the scale of fluctuation and Bartlett's distance for Keswick clay (Jaksa 1995).

spatial correlation. There appears to be a positive correlation between mean value and vertical scale

of fluctuation, indicating that the vertical spatial correlation structure of cohesionless soils would be

generally higher than that of cohesive soils. Figure 60a plots the vertical scale of fluctuation vs. the mean value of normalised cone tip resistance,

with points categorised by regional site. With the exception of the Treasure Island data (TSI), there seems

to be no site-specificity in the results. Figure 60b shows the same data, but points are categorised by

best-fit autocorrelation model: it appears that best-fit autocorrelation models are not associated to soil

type, and that none of the models provides consistently higher or lower vertical scales of fluctuation. Similar results were obtained by Uzielli et al. (2005b) while investigating random field parameters of

Robertson's soil behaviour classification index, I_c (e.g. Robertson & Wride 1998), which maps the usual (a) (b)

Figure 60. Vertical scale of fluctuation of stress-normalised cone tip resistance vs. mean value of resistance in

physically homogeneous soil unit: (a) by site; and (b) by best-fit autocorrelation model (Uzielli et al. 2005a). (a) (b)

Figure 61. Vertical scale of fluctuation of Robertson's CPT

soil behaviour classification index I_c vs. mean value of

I_c : (a) by best-fit autocorrelation model; and (b) by site (Uzielli et al. 2005b).

two-dimensional CPT-based soil classification zones onto a one-dimensional scale and is calculated

from q_{c1N} and F_R . Such parameter is a concise descriptor of soil type in terms of mechanical behaviour.

Ranges of I_c correspond to different soil behaviour zones (SBZs), with higher values (pertaining to SBZs

2 and 3) corresponding to cohesive-behaviour soils, intermediate values (SBZs 4 and 5) to intermediate

behaviour soils and lower values (SBZs 6 and 7) to cohesionless-behaviour soils. Again, the generally

stronger vertical spatial correlation structure of cohesionless soils may be seen in Figure 61.

The dependency of spatial correlation on soil type could have interesting geotechnical implications.

In the case of cone penetration testing, for instance, past research (e.g. Teh & Houlsby 1991) indicates

that the extent of the failure zone increases with increasing soil stiffness; thus, it may be postulated

that the influence zone affecting cone tip resistance is larger in sand (and a greater number of adjacent

measurements would be strongly correlated), while sleeve friction is only affected by the adjacent soil

regardless of soil type (with consequently weaker spatial correlation). Hence, the results obtained by

Uzielli et al. (2005a, 2005b) could be explained in terms of the physical phenomena involved in pene

tration. Other geotechnical testing methods could provide similar results and allow for increasingly

confident inferences; unfortunately, the literature is surprisingly limited at present.

4.5 Scale of fluctuation: literature review

A synthetic tabular review of literature values of the horizontal and vertical scale of fluctuation of a

number of geotechnical parameters is provided in Table 25. The table relies significantly on a number of

data collection efforts by Phoon et al. (1995), Jaksa (1995), Lacasse & Nadim (1996), Phoon & Kulhaw

(1999a) and Jones et al. (2002). More complete descriptions of the source data (including, for instance,

the estimation of random field parameters include Elkatheb et al. (2003b) and Uzielli et al. (2005a).

Table 25. Literature values for the horizontal (δh) and vertical (δv) scale of fluctuation of geotechnical parameters.

Property * Soil type Testing method ** δh (m) δv (m)

s u Clay Lab. testing - 0.8-8.6

s u Clay VST 46.0-60.0 2.0-6.2

q c Sand, clay CPT 3.0-80.0 0.1-3.0

q c Offshore soils CPT 14-38 0.3-0.4

1/q c Alluvial deposits CPT - 0.1-2.6

q t Clay CPTU 23.0-66.0 0.2-0.5

q c1N Cohesive-behaviour soils CPT - 0.1-0.6

q c1N Intermediate-behaviour soils CPT - 0.3-1.0

q c1N Cohesionless-behaviour soils CPT - 0.4-1.1

f s Sand CPT - 1.3

f s Deltaic soils CPT - 0.3-0.4

F R Cohesive-behaviour soils CPT - 0.1-0.5

F R Intermediate-behaviour soils CPT - 0.1-0.6

F R Cohesionless-behaviour soils CPT - 0.2-0.6

I c Cohesive-behaviour soils CPT - 0.2-0.5

I c Intermediate-behaviour soils CPT - 0.6

I c Cohesionless-behaviour soils CPT - 0.3-1.2

N Sand SPT - 2.4

w Clay, loam Lab. testing 170.0 1.6-12.7

w L Clay, loam Lab. testing - 1.6-8.7

γ' Clay Lab. testing - 1.6

γ Clay, loam Lab. testing - 2.4-7.9

e Organic silty clay Lab. testing - 3.0

σ'_p Organic silty clay Lab. testing 180.0 0.6

K S Dry sand fill PLT 0.3 -

ln(D R) Sand SPT 67.0 3.7

n Sand - 3.3 6.5

* s u = undrained shear strength; q c = cone tip resistance; q t = corrected cone tip resistance;

q c1N = dimensionless, stress-normalised cone tip resistance; f s = sleeve friction; F R = stress-normalised

friction ratio; I c = CPT soil behaviour classification index; N = SPT blow count; w = water content;

w L = liquid limit; γ' = submerged unit weight; γ = unit weight; e = void ratio; σ'_p = preconsolidation

pressure; K S = subgrade modulus; D R = relative density; n = porosity. ** VST = vane shear testing; CPT = cone penetration testing; CPTU = piezocone testing; SPT = standard

penetration testing; PLT = plate load testing.

It is possible to appreciate the strong anisotropy in spatial correlation strength, horizontal scales of

fluctuation being much larger than corresponding vertical

scales. Given the strong set-specificity of the

scale of fluctuation and its dependence on test type and estimation method, data should not be exported

uncritically to other sites.

4.6 Approximation of the variance reduction function

For a random process whose scale of fluctuation δ is known, the sample variance reduction function over

a spatial averaging interval T can be approximated, for practical applications, by (Vanmarcke 1983):

4.7 Coefficient of variation of inherent variability

For data sets satisfying the condition of weak stationarity, the dimensionless coefficient of variation of

inherent variability (Phoon & Kulhawy 1999a), η_ω , is obtained by normalising the standard deviation of

the residuals s_ω with respect to the value of the trend function at the midpoint of the homogeneous soil

unit under investigation, m_t :

where

is the standard deviation of the inherent soil variability, n is the number of data points in the profile, and

ω_i is the i -th term in the set of residuals. Implicitly, Eq. (42) assumes (as generally occurs) that residuals

are a zero-mean set.

The coefficient of variation of inherent variability is a measure of the variance of residuals, and

implicitly depends on the spatial correlation structure because it is associated with a specific trend and

the resulting set of residuals. Different values of η_ω result from the same set if different trends are

removed.

It should be noted that the mean value of the trend is

required for the calculation of $\eta \omega$. This implies that an estimated value of $\eta \omega$ is meaningful over an entire spatial extension if m_t is representative of the trend in the spatial extension itself. In other words, it is really only meaningful to estimate the coefficient of variation of inherent variability if the range in trend values in the homogeneous soil unit of interest is not too large.

While the scale of fluctuation investigates the magnitude of spatial correlation in the set of residuals,

the coefficient of variation of inherent variability is conceptually related to the magnitude of the dispersion of the residuals about the trend.

4.8 Coefficient of variation of inherent variability: literature review

The coefficients of variation reported in Table 7 (§2.14.2) are generally not calculated using the most

rigorous procedure available (i.e. following proper identification of homogeneous soil units, decompos

ition, identification of correlation structure and assessment of stationarity and application of Equations

(41) and (42)). To the writers' best knowledge, the only results in the geotechnical literature which make

use of such procedure are those reported by Uzielli (2004) and Uzielli et al. (2005a, 2005b).

Uzielli et al. (2005a) investigated the magnitude of coefficients of variation of inherent variability of

stress-normalised cone penetration parameters tip resistance in physically homogeneous soil units. As

shown in Figure 62, which reports the results for normalised cone tip resistance, cohesionless-behaviour

soil profiles are generally characterised by higher coefficients of variation. This is related to the more

erratic nature of CPT profiles in cohesionless soils, and is consistent with the physical phenomena

occurring during cone penetration.

Uzielli et al. (2005b) estimated the coefficient of variation of inherent variability for the CPT soil

behaviour classification index I_c using data from sites worldwide. Results are reported graphically in

Figure 63. The reader is referred to §4.5 for symbols and notation.

As for the scales of fluctuation reported in Figures 60 and 61, it is possible to identify a soil-type

effect in Figures 62 and 63, with cohesive-behaviour soils displaying lower values of the coefficient of

variation of inherent variability than intermediate and cohesionless-behaviour soils.

As random field parameters are bound at least to some degree to the characteristics of data (and,

hence, to the testing method), other testing methods may provide results which may be qualitatively and

quantitatively different. Further studies are warranted to allow more confident statements.

Though comparison with other literature data is not possible at present due to the absence of data con

cerning stress-normalised cone penetration testing parameters and the soil behaviour classification index,

it is to be expected that coefficients of variation obtained using the rigorous procedure should be lower

in magnitude than the ones calculated as second-moment statistics of a source data set. The latter can, in

fact, be expected to estimate the total variance of the data, thus also the variance in possible spatial trends.

4.9 Probabilistic slope stability analysis accounting for spatial variability: literature examples

The literature examples provided in §2.17 focused on reliability-based analysis using second-moment

statistics, distributions of soil properties and correlation among these, but did not account for the spatial (a) (b)

Figure 62. Coefficient of variation of inherent variability of dimensionless stress-normalised cone tip resistance

q_{c1N} vs. mean value in physically homogeneous soil unit: (a) by site; and (b) by best-fit autocorrelation model (Uzielli

et al. 2005a). (a) (b)

Figure 63. Coefficient of variation of inherent variability of the CPT soil behaviour classification index I_c vs. mean

value in physically homogeneous soil unit: (a) by best-fit autocorrelation model; and (b) by site (Uzielli et al. 2005b).

variability. When using Monte Carlo simulation, for instances, each realisation assumed that the sampled

values of soil parameters were uniform throughout the slopes. While the probabilistic analyses in §2.17

are undoubtedly a significant improvement over deterministic methods, they may overlook important

effects due to spatial averaging. Such effects can only be quantified if random field parameters are

defined as well as second-moment statistics and distributions of relevant parameters. A number of

studies have addressed the inclusion of spatial variability in slope stability analysis; in the following,

selected contributions are reported briefly. El-Ramly et al. (2003) implemented a probabilistic slope stability approach by El-Ramly et al. (2002).

Two candidate slip surfaces (shown in Figure 64) were considered: the deterministic critical slip sur

face estimated in a preliminary conventional slope

analysis; and the minimum reliability index surface obtained from a search algorithm proposed by Hassan & Wolff (1999). The Bishop method of slices was used in the model with the limit equilibrium equations re-arranged to account for the non-circular portion of the slip surface. The spatial variability of the input variables (residual friction angle of the clay-shale; peak friction angle of the sandy till; residual pore pressure ratio in the clay-shale; pore pressure ratios in the sandy till at the middle and at the toe of the slope) along the slip surface are modelled as one-dimensional random fields with an exponentially decaying correlation structure (i.e. SNX model) and a set of common horizontal autocorrelation distances (i.e. not the scale of fluctuation) selected from literature values. A lognormal distribution was selected for residual friction angle of the clay-shale; experimentally derived empirical distributions were used for the other parameters. The need to account for variance reduction was bypassed by assigning widths smaller than the autocorrelation distance to

Figure 64. Profile for the probabilistic slope stability analysis of the Syncrude Tailings Dyke by El-Ramly et al. (2003). (a) (b)

Figure 65. Results of 34000 Monte Carlo simulation realisations for probabilistic slope stability analysis by (El-Ramly et al. 2003): (a) frequency histogram; and (b) cumulative probability curve of factor of safety, considering

a horizontal autocorrelation distance of 33 m.

slices. Monte Carlo simulation was used to obtain a probability distribution of the factor of safety for reliability-based analysis.

Figure 65 shows the results of 34000 Monte Carlo simulation realisations assuming a horizontal

autocorrelation distance of 33 m, in terms of: (a) frequency histogram of the factor of safety; and (b)

cumulative probability curve of the factor of safety. Results are based on the Hassan & Wolff surface,

which provided comparatively lower factors of safety than the deterministic slip surface.

El-Ramly et al. (2003) also investigated the sensitivity of the estimated probability of the unsatisfactory

performance (i.e. the cumulative probability value corresponding to unit factor of safety) to the value of

the autocorrelation distance. It may be observed in Figure 66 that the probability of the unsatisfactory

performance increases four-fold as autocorrelation distance increases from 28 m to 38 m.

To investigate the effects of including spatial correlation, El-Ramly et al. (2003) performed simulations

without accounting for spatial variability in model parameters. The resulting probability of unsatisfactory

performance was one order of magnitude higher than the analysis accounting for spatial variability; hence,

in the specific case, spatial variability provides less conservative results.

Griffiths & Fenton (2004) investigated the effects of addressing the effects of spatial averaging in a

simulation-based probabilistic analysis. The method of analysis is essentially that referred to in §2.17;

however, the reference lognormal probability distribution from which values of normalised strength are

sampled is modified accounting for spatial averaging. The main effect on the distribution is perhaps the

reduction of both the log-mean and the log-variance of the

distribution.

Figure 66. Variation of the probability of unsatisfactory performance with autocorrelation distance (El-Ramly et al. 2003).

5 IDENTIFICATION OF PHYSICALLY HOMOGENEOUS SOIL UNITS

The physical homogeneity of soil units (HSU) is a fundamental prerequisite for variability analyses.

Performing variability analyses on soils which are not homogeneous in terms of the property of interest

can result in incorrect estimates. The first issue which is relevant for physical homogeneity assessment is perhaps whether such assess

ment should occur on a subjective or an objective basis. A purely subjective assessment would rely

uniquely on expert judgment. It has been observed (e.g. Hegazy & Mayne 2002) that, in practice, such

approach does not provide optimum results. A purely objective assessment (i.e. based exclusively on

numerical criteria) is not feasible in practice, as at least the check on data quality and the definition

of relevant parameters requires geotechnical background on the part of the user. Hence, in practice,

homogeneity assessment should be both subjective and objective to some degree, and should include an

efficient numerical analysis based on a suitable, expert choice of reference parameters. A major issue to be addressed refers to whether the homogeneity should be assessed in terms of

soil composition or soil behaviour. Research has shown that these do not display a one-to-one corres

pondence (e.g. Hegazy & Mayne 2002; Zhang & Tumay 2003). Soils which may be homogeneous in

terms of composition may not be so in terms of mechanical behaviour. Hence, one should define and

report the criteria adopted for physical homogeneity assessment. The in-situ penetration resistance of a compositionally homogeneous soil layer, for instance, increases with depth due to overburden stress and other in-situ conditions. Most geotechnical testing methods provide direct information regarding the mechanical response of soil to penetration. Hence, homogeneity is usually assessed, in a quantitative perspective, on the basis of soil behaviour rather than composition. However, it may of interest to investigate variability in terms of the latter. Hight & Leroueil (2003) provided the example of Bothkennar clay, for which three facies (mottled, bedded and laminated as shown in Figure 67; each with a distinctive fabric and associated with a distinct depositional environment) have been recognized despite a very uniform composition (as shown in Figure 68). The influence of the fabric and depositional environment on the mechanical properties was observed experimentally (Figure 69). Hence, despite a compositional physical homogeneity, the geomaterial cannot be considered physically homogeneous from the point of view of mechanical behaviour. Variability may also occur due to man-induced factors. Hight & Leroueil (2003) showed the effects of different forms of damage and/or disturbance to samples from Bothkennar clay in terms of triaxial compression tests (Figure 70). Such variability in mechanical behaviour adds to the natural variability discussed earlier.

5.1 Assessment of physical homogeneity: literature review

In the following, a selection of methods making use of both subjective and objective criteria for the

identification of physically homogeneous soil units is

presented. The review in no way aims to be

exhaustive.

Figure 67. Facies of the Bothkennar clay (Hight & Leroueil 2003)

Figure 68. Grain size distribution of Bothkennar clay (Hight & Leroueil 2003).

Figure 69. Non-homogeneity in mechanical behaviour

of Bothkennar clay (Hight & Leroueil 2003). Figure 70. Effects of damage and/or disturbance to Bothkennar clay samples (Hight & Leroueil 2003).

5.1.1 Probabilistic-fuzzy stratigraphic delineation

Zhang & Tumay (2003) summarised the findings of their own previous research focusing on the CPT

based identification of physically homogeneous soils through a probabilistic and fuzzy methodology.

The approach is based on the consideration that there is no univocal correspondence between existing (a) (b)

Figure 71. Identification of homogeneous units from soil classification for the NGES/Texas A&M site:

(a) probability profile; (b) fuzzy soil type index profile (Zhang & Tumay 2003).

soil composition and the mechanical resistance to penetration, and tries to quantify the uncertainty in

the composition-behaviour correspondence. The method is based upon a single soil classification index,

obtained through a coordinate change procedure from existing CPT-based classification charts. At each

depth where measurements are available, the method provides both a probabilistic and a fuzzy estimate

of soil type in terms of behaviour. In the first case, the probability of a soil being “sandy”, “silty” or

“clayey” is returned (with the sum of the three probabilities amounting to unity) on the basis of the soil

classification index U . Fuzzy estimation provides a degree of membership (from 0: no membership to

1: full membership) of a soil datum (in this case U) to the fuzzy sets of “highly probably sandy” (HPS),

“highly probably medium” (HPM) and “highly probably clayey” (HPC). The sum of membership values

need not be unity. Figure 71 (Zhang & Tumay 2003) illustrates the application of the method to data from the NGES/Texas

A&M site. The approach can be integrated by the calculation of the intraclass correlation coefficient

(e.g. Wickremesinghe & Campanella 1993; Cherubini et al. 2006) on the soil classification index to

allow for the statistical identification of probable stratigraphic interfaces.

5.1.2 Statistical clustering

Hegazy & Mayne (2002) proposed a method for soil stratigraphy delineation by cluster analysis of

piezocone data. The objectives of cluster analysis applied to CPT data were essentially: (a) objectively

define similar groups in a soil profile; (b) delineate layer boundaries; and (c) allocate the lenses and

outliers within sub-layers. Visual inspection of data (e.g. OCR profile and index properties in Figure 72) may allow delineation of

the main interfaces between homogeneous layers. Cluster analysis, on the contrary, allows identification

of stratigraphic boundaries at various levels of resolution: if a configuration with a low number of clusters

is referred to (e.g. 5 cluster-configuration in first column from left in Figure 73), the main boundaries are

identified; with increasing number of clusters, greater resolution is achieved, by which less pronounced

discontinuities are captured.

Figure 72. Identification of primary stratigraphic boundaries from cluster analysis - Recife, Brazil site (Hegazy &

Mayne 2002).

Figure 73. Identification of homogeneous soil units from clustering analysis: comparative configurations for

variable number of clusters on same profile of corrected cone tip resistance - Recife, Brazil site (Hegazy & Mayne 2002).

Hegazy & Mayne (2002) and Facciorusso & Uzielli (2004) assessed the very good capabilities of

clustering techniques for CPT-based soil stratigraphy delineation. However, they also observed that

clustering is computationally expensive, and is difficult to implement for large data sets.

5.1.3 Modified Bartlett's statistic method

Phoon et al. (2003c) proposed a statistical moving window methodology for the identification of phys

ically homogeneous soil units using Bartlett's statistic. The sampling window is divided into two equal 0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32 0 100 200 0 2 4 6 8 q t (MPa) B stat I III IV II Depth (m)

Figure 74. Example identification of potentially homogeneous soil units I-IV based on Bartlett statistic profile

(bottom) from corrected cone tip resistance data (top) (Phoon et al. 2004).

segments and the sample variance is calculated from data points lying within each segment. For the case

of two sample variances, σ^2_{B1} and σ^2_{B2} , the Bartlett test statistic reduces to

where m_B is the number of data points used to evaluate σ^2

B_1 and σ^2_{B2} . The total variance, σ^2_B , is defined as

The constant C_B is given by

Bartlett's statistic indicates the difference between the sample variances σ^2_{B1} and σ^2_{B2} in the two adjacent

segments. The value of the Bartlett statistic increases with increasingly different sample variances, and

is equal to zero if the sample variances are equal. A continuous Bartlett statistic profile can be generated

by moving a sampling window over a spatially ordered data profile. Peaks in the Bartlett statistic profile

indicate possible interfaces between physically homogeneous soil units. Figure 74 (Phoon et al. 2004) shows an application of the methodology to corrected cone tip resistance

data. Phoon et al. (2003c) stated that the identification process based on the modified Bartlett statistic

profile is only a preliminary assessment, and that engineering judgement should be exercised to ensure

that results are physically meaningful. Hence, the Bartlett statistic profile is to be interpreted only in a

qualitative way in the context of soil boundary identification.

5.1.4 Statistical moving window

Uzielli (2004) proposed a statistical moving window procedure to identify physically homogeneous soil

units making use of coefficients of variation of selected properties. Each moving window is made up

of two semi-windows of equal height above and below a centre point. A suitable height of the moving

window of $W_d = 1.50$ m was established by Uzielli (2004) on the basis of past research (e.g. Lunne et al.

1997) and calibration of results with available borehole logs. At each centre point (identified by its depth

z_c), the mean, standard deviation and coefficient of variation (COV) were calculated for data lying in the

interval $z_c - W_d/2 \leq z \leq z_c + W_d/2$, corresponding to the upper and lower limits of the moving window.

Figure 75. Application of the moving window procedure to cone penetration testing data (Uzielli 2004).

HSUs are essentially identified by delineating soundings into sections where the COV of one or more

user-selected parameters are less than a preset threshold. Harr (1987) proposed a value of 0.1 for COV

as the upper limit for “low dispersion” in soil properties. A minimum width for homogeneous units can

be preset by the user.

Uzielli et al. (2005a) applied the procedure to CPT soundings using the normalised, dimensionless

cone penetration resistance q_{c1N} , the normalised friction ratio F_R and the soil behaviour classification

index, I_c . These parameters have been referred to in previous sections of the paper.

Figure 75 illustrates the application of the method to cone penetration testing data. The coefficients

of variation of the logarithm of normalised cone tip resistance and soil behaviour classification index

are plotted in the two rightmost diagrams. The spatial extension in which they are both smaller than 0.10

(for at least 4.50 m consecutively) is identified as a physically homogeneous soil unit.

In principle, the procedure can be applied to data from other in-situ tests, provided that a parameter

which is related to soil classification is available. In the case of flat dilatometer testing (DMT), for

instance, the material index I_D is a suitable parameter. Figure 76 shows an application of the method to

DMT data, where the coefficient of variation of I_D is the reference parameter.

5.2 Second-moment representation of homogeneous soil units

The distribution of values of a soil property in a physically homogeneous soil unit can be represented

concisely in a second-moment sense by a mean and a standard deviation. In Figure 77 (Uzielli et al.

2005a), normalised friction ratio and normalised cone tip resistance data from 70 physically homoge

neous soil units are plotted on Robertson's $F_R - q_{c1N}$ chart through their second-moment statistics: each

point represents one unit, with the circles located at the mean values and error bars indicating plus and

minus one standard deviation.

5.3 Assessment of identification methods

The performance of the homogeneous soil unit identification methods should be assessed using geotech

nical knowledge or statistical criteria. A variety of soil classification charts based on in-situ tests are

available in the geotechnical literature; these provide an effective means for qualitative assessment of

Figure 76. Application of the moving window procedure to flat dilatometer testing data.

Figure 77. Representation of second-moment data from 70 physically homogeneous soil units on Robertson's

CPT-based $F_R - q_{c1N}$ chart (Uzielli et al. 2005a).

the homogeneity of a data set. If data points from a soil unit plot as a well-defined cluster, homogeneity

can be expected. For CPTU testing, for instance, Robertson's (1990) classification charts were used by

Phoon et al. (2004), Hegazy & Mayne (2002) and Uzielli et al. (2005a) to assess the performance of the

methods they proposed (see Figure 78, Figure 79 and Figure 80). For DMT testing, Marchetti & Crapps'

(1981) soil classification chart can be used (e.g. Figure 81).

Figure 78. Assessment of the Bartlett statistic method: plotting data from a physically homogeneous soil unit on

Robertson's CPTU-based $B_q - q_n$ chart (Phoon et al. 2004).

Figure 79. Assessment of statistical clustering method: plotting data from a physically homogeneous soil unit on

Robertson's CPTU-based $B_q - Q$ chart (Hegazy & Mayne 2002).

Uzielli et al. (2005a) also employed a statistical criterion to assess the performance of the moving

window statistical methodology presented in §5.1.4. They calculated the coefficient of variation of

the soil behaviour classification index I_c (which is an efficient descriptor of soil behaviour) in each

identified homogeneous soil unit. A low coefficient of variation indicates low data dispersion and,

hence, homogeneity. The procedure was shown to perform very well both in cohesive and non-cohesive

behaviour soil units, as the average of the coefficients of variation was 0.02, with values from individual

soil units never exceeding 0.10 (a value indicating low dispersion according to Harr's rule of thumb).

Figure 80. Assessment of the statistical moving window

method: plotting data from two physically homogeneous

soil units (CHS: cohesive-behaviour; CHL: cohesionless

behaviour) on Robertson's CPT-based $F_R - q_{c1N}$ chart (Uzielli

et al. 2005a). Figure 81. Assessment of the statistical moving window method: plotting data from a physically homogeneous soil unit on Marchetti and Crapps' (1981) $I_D - E_D$ chart.

6 ASSESSMENT OF WEAK STATIONARITY

Stationarity is an important prerequisite for stochastic modelling of soil properties because the statistical

procedures employed are based on the assumption that data samples consist of stationary observations. Among stochastic processes, strictly stationary processes are characterised by the fact that all finite

dimensional probability distributions are invariant to translation. As a consequence, all of their statistical

properties (e.g. mean, variance, skewness, kurtosis, etc.) are spatially invariant. Strict stationarity is a

severe requirement in practice and mostly of conceptual importance only. In practice, it is necessary and

convenient to relax the constraint of strict stationarity and refer to weak stationarity. A process is said to

be stationary in the second-order sense (or weakly stationary) if: a) its mean is constant (i.e. there are no

trends in the data); b) its variance is constant; and c) the correlation between the values at any two points

at distinct spatial locations, depends only on the interval in the index set between the points, and not

on the specific spatial location of the points themselves, i.e. autocorrelation is a function of separation

distance only. In the case of laboratory or in-situ geotechnical testing, stationarity can only be identified

in a weak or second-order sense because of the limitations in sample size.

6.1 Testing for weak stationarity

There are two general approaches to statistical testing for stationarity; parametric and nonparametric.

Parametric approaches require the formulation of hypotheses regarding the nature and distribution of

the data set under investigation. Non-parametric

approaches, on the contrary, do not make any basic assumptions about the nature of the system, e.g. they are not based on the knowledge or assumption that the population is normally distributed. By making no assumptions about the nature of the data, non-parametric tests are more widely applicable than parametric tests which often require normality in the data. While more widely applicable, the trade-off is that non-parametric tests are less powerful than parametric tests. As stationarity is tested by the use of statistical procedures, it is perhaps useful to remark that assessment is referred to a specified confidence level. It should be emphasised (e.g. Baecher & Christian 2003) that stationarity is an assumption of the model, and may only approximate the real-world situation. Also, stationarity usually depends upon

Figure 82. Dependency of weak stationarity from scale of investigation (Fenton 1999a).

Figure 83. Sample realisation with apparent non-stationarity in the mean (Phoon et al. 2004).

scale. Phoon et al. (2003c), recalling the fractal behaviour of soil properties, specified that even weak stationarity cannot be verified in a strict sense over a finite length because longer scale fluctuations could be mistakenly identified as a non-stationary component (e.g. trend). Hence, at a small scale such as a construction site, soil properties may behave as if drawn from a stationary process; whereas, the same properties over a larger region may not be.

The dependence of weak stationarity from the scale of investigation is shown in Figure 82 (Fenton 1999a), in which the process shows a clear trend over the spatial extension D (and is therefore non

stationary). However, if the scale of investigation is increased, the process could be part of a larger

scale stationary process (the fluctuating line) or of another non-stationary process (the quasi-monotonic

line with a negative trend). Figure 83 (Phoon et al. 2004) shows a realisation displaying an apparent

non-stationarity in the mean (i.e. a spatial trend) though it was simulated from a stationary model.

In the following, two stationarity assessment procedures are proposed. The first procedure is referred

to as Kendall's tau test. Among the classical statistical tests which have been used in the geotechnical

literature, Kendall's tau test has been identified (e.g. Jaksa 1995) as the most powerful for geotechnical

applications, though recognising that, in some instances, the test failed to reject apparently non-stationary

data for moderately small samples. Kendall's tau test (e.g. Daniel 1990) is a non-parametric test for statis

tical independence. As statistical independence implies stationarity (the converse is not true), Kendall's

test can be used for assessing the latter.

Kendall's test, as other classical stationarity tests (e.g. statistical runs test, Spearman's rank coeffi

cient), is based on the assumption of spatially uncorrelated input data. This assumption is antithetical

to the fundamental premise in geotechnical variability analysis, namely the existence of a correlation

structure in spatially varying soil properties. Due to the resulting bias in autocorrelation coefficients in

the correlated case, the application of classical statistical tests may result in unconservative assessments,

i.e. non-stationary sets may be erroneously classified as weakly stationary. Note that Kendall's test (or other classical parametric or non-parametric tests) is not

strictly applic

able to correlated data. To the writers' knowledge, the only non-parametric method that is developed for

correlated data is the boot-strapping method presented by Phoon & Fenton (2004) and Phoon (2006a).

It is difficult to develop a non-parametric method for correlated data because minimal assumptions

are imposed not merely on probability distribution, but on the correlation structure as well. In con

trast, classical non-parametric methods for independent data only minimize assumptions on probability

distributions. The procedures reported in the following implicitly neglect measurement and random testing errors,

i.e. they assume that measured values reflect the true values of the parameter of interest. This assumption

is acceptable for tests which have been shown to be largely operator-independent and to have very low

random measurement errors such as the CPT or DMT. Results of tests with high measurement and

random uncertainty (such as the SPT, which also has a very large measurement interval) are not reliable

inputs for the method.

6.1.1 Kendall's tau test

Kendall's test involves the calculation of the test statistic, τ_{ken} , which measures the probability of con

cordance minus the probability of discordance between measurements in a data set. The test statistic is

given by:

where n_d is the number of pairs of observations in a data set; and

in which P_{ken} is the number of concordant pairs of observations and Q_{ken} is the number of discordant

pairs of observations. To calculate such parameters, a total of $(1/2)n d (n d - 1)$ comparisons are made

between all possible pairs of observations. A pair of observations (x_i, y_i) , (x_j, y_j) is said to be concordant

if $x_i > x_j$ and $y_i > y_j$, while it is said to be discordant if $x_i > x_j$ and $y_i < y_j$. A visual representation of

concordant and discordant data pairs is provided in Figure 84. The values of τ_{ken} range from -1 to +1, indicating, respectively, perfect negative and positive

correlation. A value close to zero indicates low correlation. For $n d \leq 40$, critical values of τ_{ken} for rejecting the null hypothesis of independence are available in

tabulated form (e.g. Daniel 1990). For $n d > 40$, the following statistic is calculated from τ_{ken} :

The statistic $z_{\tau,ken}$ is normally distributed with zero mean and unit standard deviation; hence, it can be

compared to values in standard normal distribution tables. For a 95% confidence level, for instance, if

$|z_{\tau,ken}| \leq 1.96$ the data can be assumed to be statistically independent. concordant discordant

Figure 84. Concordant and discordant pairs of observations.

6.1.2 The MBSR method

A procedure based on Bartlett's statistic was first proposed by Phoon et al. (2003c) with the aim of

providing a more rational basis for rejecting the null hypothesis of stationarity in the correlated case as

it incorporates the correlation structure in the underlying data and as it includes all fundamental phases

of random field modelling analysis (stationarity, choice of trend function and autocorrelation model).

The method was subsequently integrated by restrictive conditions on the fit of autocorrelation models

by Uzielli (2004); the resulting procedure is referred to hereinafter as MBSR (i.e. Modified Bartlett

Statistic Revised).

MBSR requires the preliminary calculation of the Bartlett statistic profile (see §5.1.3), the estimation

of the scale of fluctuation by fitting theoretical autocorrelation models (see §4.3.1) and the subsequent

comparison between the maximum value of the Bartlett statistic and a critical value. The latter is calcu

lated as follows. First, the profile factors in Table 26 are defined. In Table 26, δ is the scale of fluctuation;

z is the measurement interval; L_d is the spatial length of the soil record of length; and W_B is the spatial

extension of half the sampling window.

The number of points in one scale of fluctuation, k_B , should lie between 5 and 50. The normalised

sampling length, I_1 , is also taken to lie between 5 and 50. Phoon et al. (2003c) suggested that the

normalised segment length be chosen as $I_2 = 1$ (for $k_B \geq 10$) and $I_2 = 2$ (for $k_B < 10$).

Once the Bartlett statistic profile is obtained, the peak value B_{max} is identified. Critical values (see

Table 27) of this modified Bartlett test statistic at 5% level of significance for several autocorrelation

models, among which single exponential (SNX), cosine exponential (CSX), second-order Markov (SMK)

and squared exponential (SQX), were provided by Phoon et al. (2003c) using the spectral approach by

Shinozuka & Deodatis (1991). The null hypothesis of stationarity in the variance is rejected at 5% level

of significance if $B_{max} > B_{crit}$.

Phoon et al. (2003c) observed that the stationarity

rejection criterion is significantly dependent on

the ACM employed. The criterion is most strict for SNX and becomes increasingly relaxed in the order

of CSX, SMK and SQX.

Following application of the MBSR method, three outcomes are possible:

1. MBSR is not applicable in its entirety, due to one or more of the following: a) the minimum value of

the determination coefficient $R^2_{\min}$ is not reached; b) one or more of the dimensionless profile factors

of the MBS procedure (k_B , I_1 , I_2), which depend on the scale of fluctuation, δ , sample spacing, Δz ,

and sample size, n_B , falls outside the ranges established by Phoon et al. (2003c);

2. MBSR can be applied, but $B_{\max} \geq B_{\text{crit}}$; thus, the profile is classified as non-stationary at 5%

significance level;

3. MBSR procedure can be applied in its entirety, and at least one ACM provided: a) stationarity at the

5% level; and b) a coefficient of determination $R^2 \geq R^2_{\min}$. If more than one ACM satisfies the above

conditions, the scale of fluctuation deriving from the ACM with the maximum R^2 is adopted. Table 26. Profile factors for the Bartlett statistic-based method (Phoon et al.

2003c). Profile factor Expression Number of points in one scale of fluctuation $k_B = \delta / \Delta z$ Normalised sampling length $I_1 = L_d / \delta$ Normalised segment length $I_2 = W_B / \delta$ Table 27. Critical modified Bartlett test statistics at 5% level of significance for SNX, CSX, SMK and SQX autocorrelation models (from Phoon et al. 2003c). ACM I_2 SNX $1 B_{\text{crit}} = (0.23k_B + 0.71) \ln(I_1) + 0.91k_B + 0.23$ SNX $2 B_{\text{crit}} = (0.36k_B + 0.66) \ln(I_1) + 1.31k_B - 1.77$ CSX $1 B_{\text{crit}} = (0.28k_B + 0.43) \ln(I_1) + 1.29k_B - 0.40$ SMK $1 B_{\text{crit}} = (0.42k_B - 0.07) \ln(I_1) + 2.04k_B - 3.32$ SQX $1 B_{\text{crit}} = (0.73k_B - 0.98) \ln(I_1) + 2.35k_B - 2.45$

Figure 85. Output of application of MBSR to

a normalised cone tip resistance profile (Uzielli

2004). Figure 86. Comparative assessment of weak stationarity (Y) or non-stationarity (N) by MBSR versus KTT (Uzielli et al. 2004). Thus, a given value of δv calculated from one of the ACMs is accepted only if: a) it refers to a

weakly stationary fluctuating component [i.e. $B_{max} < B_{crit}$ in the MBSR procedure]; b) the determination

coefficient, R^2 , of the fit of the ACM to the significant portion of the autocorrelation function (defined

by the parameter r_B) does not fall below a user-defined threshold. An example of MBSR-based weak stationarity assessment for a profile of normalised cone tip

resistance in a homogeneous soil unit is shown in Figure 85.

6.1.3 Comparison with results of Kendall's tau test for statistical independence

To verify the severity of the MBSR test compared to a statistical test which assumes uncorrelated input

data, a comparative assessment of stationarity with Kendall's tau test at 95% confidence level was

performed by Uzielli et al. (2004) on detrended profiles of normalised cone tip resistance. Figure 86

compares stationary assessment using MBSR and KTT. Both tests agree in the majority of the cases

(open diamond and circle). However, HSUs that are MBSR stationary are rarely not KTT-stationary

(solid triangles). Conversely, there are quite a number of HSUs that are KTT-stationary, but not MBSR

stationary (solid squares). If one assumes that MBSR provides more accurate stationarity assessments,

the power of KTT (rejecting the null when it is false) is lower, i.e. KTT is less discriminatory. However, the

performance of the tests is comparable. On the basis of such findings, it would appear that Kendall's test

can be used with sufficient confidence. More investigations are warranted to support these conclusions.

7 ADVANCED MODELLING OF SPATIALLY RANDOM GEOTECHNICAL SYSTEMS

The description of a random field in the second-moment sense through a mean, standard deviation,

a scale of fluctuation and a spatial correlation function is useful to characterise a spatially variable soil

property. Moreover, it allows the spatial averaging effect to be investigated, which significantly affects

quantitative variability assessment. However, if the results of random field modelling are to be used in reliability-based engineering

applications, some possible limitations in this approach should be recognised. For instance, it could be suspected that, if spatial variability of soil properties is included in an

engineering model, stresses and/or displacements which would not appear in the homogeneous case (i.e.

in which variability is not addressed) could be present. Hence, addressing spatial variability may allow

a more realistic modelling of the behaviour of geotechnical systems. One of the most important benefits of random field modelling is the capacity to simulate data series.

By using sets of random field simulations and implementing the variability in non-linear finite element

meshes, the Monte Carlo technique can be used to predict reliability of geotechnical systems with

spatially variable properties.

A number of studies have focused, in recent years, on the combined utilisation of random fields, non

linear finite element analysis and Monte Carlo simulation for investigating the behaviour and reliability

of geotechnical systems when the variability of soil properties which are relevant to the main presumable

failure mechanisms is considered. A number of these are reviewed in the following.

Though the aforementioned studies address a wide variety of geotechnical systems, a number of

important general observations can be made on the basis of their results. First, when soils are mod

elled as spatially variable, the modelled failure mechanisms are quite different - and significantly more

complex - than in the case of deterministic soil properties. One example is that a footing resting on

a spatially varying medium will fail realistically on one side, in contrast to the classical symmetrical

Prandtl bearing capacity failure pattern. Another simple example is that realistic local slips along a long

embankment can be reproduced. None of these behaviours can be simulated by FEM with conventional

homogeneous or stratified soil profiles. More in-depth examples are given by Kim (2005) and Kim &

Santamarina (2006). Second, there generally exists a critical correlation distance which corresponds to

minimum reliability. Third, phenomena governed by highly non-linear constitutive laws are affected the

most by spatial variations in soil properties.

Hence, variance reduction alone is unable to convey a comprehensive picture of the implications of

spatial variability on the behaviour of a geotechnical system. Both statistical variability (i.e. in second

moment sense) and spatial variability (i.e. spatial correlation) of soil properties affect the reliability of

geotechnical systems. Three-dimensional aspects are also relevant in some cases.

The number of studies making use of random field simulation, finite elements and Monte Carlo simu

lation is still limited. Important assumptions such as that of anisotropy in spatial correlation structures

are not always addressed, and the selection of values for probabilistic descriptors of the random fields

are often assumed from literature. Moreover, such approaches are not conducted, at present, at a routine

level. Nonetheless, the importance of the results they are suggesting should serve as a stimulus for future

research and for the transposition of results to a more reproducible and user-friendly perspective.

7.1 Slope stability

Griffiths & Fenton (2004) proposed the application of the Random Finite Element Method (RFEM) to

slope stability analysis. The RFEM (e.g. Griffiths & Fenton 1998) accounts for spatial correlation and

spatial averaging effects, and, in contrast to probabilistic approaches such as those described in §4.9,

does not require a priori assumptions regarding the shape or location of the failure mechanism. Hence,

the failure mechanism is able to seek its way through the modelled soil volume: this is significantly

more consistent with the physical reality of a spatially variable soil. Figure 87 shows two possible spatial

distributions of normalised strength corresponding to two values of the normalised scale of fluctuation

(which provides a measure of the magnitude of spatial correlation of normalised strength) for the simple

model slope referred to in §2.17 and §4.9. Darker areas correspond to higher normalised undrained shear

strength. It is unlikely, for instance, that the mesh with weaker spatial correlation (top) may develop a

slip surface similar to the configuration with stronger spatial correlation (bottom). RFEM analysis is

able to capture the diversity between the modes of failure of weakly and strongly spatially correlated

profiles (top and bottom, respectively, of Figure 88).

Figure 89 shows the probability of failure versus the coefficient of variation of the lognormal random

field for various values of the normalised scale of fluctuation. The existence of a crossover point, which

indicates a change in the relationship between coefficient of variation, probability of failure and factor

of safety, may be observed.

In comparison with the results of the spatially averaged Monte Carlo simulation described in §4.9,

and for any given mean of the lognormal random field, RFEM yields lower probabilities of failure for

low coefficients of variation and higher probabilities of failure for higher coefficients of variation. This

is shown in Figure 90.

In synthesis, Griffiths & Fenton (2004) stated that simplified probabilistic analyses, in which spatial

variability is not accounted for, could lead to unconservative estimates of reliability. This effect is most

evident at lower factors of safety or for random fields of undrained strength with high coefficients of

variation.

7.2 Seepage through water retaining structures

Griffiths & Fenton (1998) investigated the influence of the variability in soil permeability on the second

moment statistics of the exit gradient at the downstream side of a water-retaining earth structure in both

two and three dimensions. Figure 91 illustrates the difference in geometry in a typical flow net between

Figure 87. Influence of normalised scale of fluctuation
on spatial variation of soil properties for RFEM analysis

(Griffiths & Fenton 2004). Figure 88. Examples of failure
modes for weakly (top) and strongly (bottom) spatially
correlated configurations of the model slope using RFEM
(Griffiths & Fenton 2004).

Figure 89. Probability of failure versus coefficient of
variation for various normalised scales of fluctuation:

results of RFEM analysis (Griffiths & Fenton 2004). Figure
90. Comparison of the probability of failure for various
coefficients of variation of normalised undrained strength
predicted by RFEM (curves with points) and by probabilistic
analyses accounting for spatial averaging (Griffiths &
Fenton 2004).

Figure 91. Typical geometry of flow nets for: (a)
deterministic soil permeability; and (b) spatially random
soil

permeability (Griffiths & Fenton 1998).

(a) the deterministic permeability; and (b) spatially
variable permeability cases. A lognormal distribution

was selected for the random field of permeability. The
results of the two-dimensional analysis are illustrated in
Figure 92: (a) shows the mean value of

exit gradient versus the coefficient of variation for
different values of the scale of fluctuation; (b) shows

the mean value of exit gradient versus the scale of
fluctuation for various coefficients of variation; (c)

shows the standard deviation of exit gradient versus the
coefficient of variation for different values of (a) (b)
(d)(c)

Figure 92. Results of 2D analysis of impact of variability
in permeability on exit gradient from water retaining earth
structures (Griffiths & Fenton 1998).

the scale of fluctuation; (b) shows the standard deviation

of exit gradient versus the scale of fluctuation

for various coefficients of variation.

The corresponding plots for the three-dimensional case are shown in Figure 93. In both cases, Grif

fiths & Fenton (1998) inferred that the inclusion of variability in permeability provided results that were

significantly different from the deterministic case. First, the mean gradient remains essentially constant

around the deterministic value for small scales of fluctuation, but tends to increase for larger scales of

fluctuation (see Figure 92a for the 2D case; Figure 93a for the 3D case). The standard deviation, on the

other hand, increases for all ranges of the scale of fluctuation (see Figure 92c for the 2D case; Figure 93c

for the 3D case). Second, there appears to be a worst-case value of the scale of fluctuation, i.e. a critical

value for which both the mean and the standard deviation of the exit gradient reach a local maximum at

the same time (see Figure 92b, Figure 92d for the 2D case; Figure 93b Figure 93d for the 3D case).

Griffiths & Fenton (1998) compared the 2D and 3D results, and observed that the 3D case allows

the flow greater freedom to avoid the low permeability zones, increases the averaging effect within each

realisation and reduces the overall randomness of the results observed between realisations. Results

were also interpreted in the context of reliability-based design, and a relationship between the traditional (a) (b) (d)(c)

Figure 93. Results of 3D analysis of impact of variability in permeability on exit gradient from water retaining earth

structures (Griffiths & Fenton 1998).

factor of safety and the probability of failure was

established. Figure 94 shows the results of 1000 Monte Carlo simulations in the 2D case for the critical scale of fluctuation of 2 m and a unit coefficient of variation of the lognormal random field of permeability. It is seen that the results of simulations are very well approximated by a lognormal distribution, whose parameters may be recorded. Figure 95 shows the curves obtained by plotting the probability that the probabilistic exit gradient exceeds the deterministic exit gradient versus the coefficient of variation of the lognormal random field of permeability, both for the 2D and 3D cases and for a scale of fluctuation of 2 m. Such plot allows inference regarding the degree of conservatism in the deterministic prediction.

7.3 Stability of underground pillars

Griffiths et al. (2002) studied the influence of spatially varying strength on the failure of axially loaded rock pillars commonly used in underground construction and mining processes. Rock strength was characterised by its unconfined compressive strength using an elastic-perfectly plastic Tresca failure criterion. An example of deformed mesh at failure is provided in Figure 96a. The spatial variability of strength (modelled as a lognormal random field with a single exponential-type correlation structure) can be appreciated, as well as the complexity in the failure mechanism resulting from the heterogeneity in strength itself. The main conclusion by Griffiths et al. (2002) was that rock strength variability can significantly reduce the compressive strength. More specifically, as the coefficient of variation of the rock strength increased, the expected rock strength decreased (see Figure 96b). Also, while the decrease in compressive

strength was greatest for small scales of fluctuation, the existence of a critical scale of fluctuation, which

yielded the most significant reduction in strength, was observed (see Figure 96c). Figure 96d shows that

Figure 94. Histogram of exit gradients in the 2D case

for a scale of fluctuation of 2 m and unit coefficient of

variation (Griffiths & Fenton 1998). Figure 95. Probability that the probabilistic exit gradient exceeds the deterministic exit gradient (2D and 3D cases) for a scale of fluctuation of 2 m (Griffiths & Fenton 1998).

increases in coefficient of variation of strength results in an increase in the coefficient of variation of the

bearing capacity factor; such positive relationship is stronger for higher values of the scale of fluctuation

due to the spatial averaging effect.

7.4 Soil-structure interaction

Soil variability is a major source of soil-structure interaction problems. Breysse et al. (2005) showed

the influence of spatial variability on soil-structure interaction for a number of simplified systems. For

instance, they investigated the ratio of differential to absolute settlement variance for two neighbouring

footings behaving independently (i.e. structural stiffness is neglected) loaded with the same load and

resting on an elastic, spatially heterogeneous soil whose local stiffness is modelled using a Winkler

model. The spatial variability is defined by the correlation length of the Winkler-spring stiffness (see

Figure 97).

Figure 98 shows the ratio of differential to absolute settlement variance (obtained by Monte Carlo

simulation) as a function of the distance between footings

(L) for $B = 1.5$ m and for different correlation

lengths ranging from 1.5 m to 1000 m. It may be seen that, for a given distance between footings, larger

correlation distances result in statistically smaller differential settlements.

Figure 99 shows the same ratio of differential to absolute settlement variance, but as a function of

the ratio of correlation length to footing size, and for various ratios of distance to footing size. For any

value of the ratio of distance to footing size, the curve is bell-shaped, i.e. there is a range of correlation

lengths which corresponds to statistically worst consequences.

Breysse et al. (2005) also investigated the effects of structural stiffness in soil-structure interaction

accounting for spatial heterogeneity. A fully coupled soil-structure analysis was performed to investigate

the behaviour of a pile groups linked by a more or less rigid slab. In the model, illustrated in Figure 100,

each pile is modelled by a spring whose elastic stiffness is assumed to vary randomly. The slab has a

finite flexural stiffness, and is loaded with a uniform load.

Figure 101 shows the effect of the slab depth/stiffness ratio on the load on the central pile for a constant

correlation distance of pile stiffness of 10 m. The 5%, 50% and 95% fractile curves are obtained by Monte

Carlo simulation. Figure 102 illustrates the effect of varying correlation distance, all other parameters

constant. As in the previous case, there is a range of correlation distances (around 2 m) which result in

a greater variability and, also, in a higher pile load.

In both cases, therefore, the existence of a critical

correlation distance, i.e. one yielding statistically

less favourable conditions, was verified. While a detailed discussion on the physical significance of the

critical values for the models specifically investigated is provided in the source reference, it is sufficient (a) (b) (c) (d)

Figure 96. Selected results from the underground pillar reliability analysis (Griffiths et al. 2002).

Figure 97. Reference model for the analysis of the effects of spatial variability on the settlements of two independent

footings (Breysse et al. 2005).

Figure 98. Ratio of differential to absolute settlement variance as a function of the distance between footings D and

correlation length L_c . (Breysse et al. 2005).

Figure 99. Ratio of differential to absolute settlement variance as a function of ratio of correlation length to footing

size, L_c/B , and ratio of distance to footing size, L/B (Breysse et al. 2005).

Figure 100. Model of the pile-cap system (Breysse et al. 2005).

here to acknowledge the importance of accounting for spatial variability in soil-structure interaction

analyses.

Popescu et al. (2005a) investigated the differential settlements and bearing capacity of a rigid strip

foundation on an overconsolidated clay layer. The undrained strength of the clay was modelled as a non

normal random field. The deformation modulus was assumed to be perfectly correlated to undrained

shear strength. Ranges for the probabilistic descriptors of the random field (coefficient of variation,

horizontal and vertical correlation distances, distribution functions) were assumed from the literature

(a symmetric beta distribution and a gamma distribution were used comparatively). Overall settlements

(including uniform and differential settlements) were computed using non-linear finite elements in a

Monte Carlo simulation framework. The reference finite element mesh used in the study is shown

Figure 101. Effect of soil variability and stiffness ratio λ on pile load (Breysse et al. 2005).

Figure 102. Effect of correlation length L_c and soil variability on pile load for central pile (Breysse et al. 2005).

Figure 103. Reference finite element mesh and boundary conditions for the rigid strip foundation model (Popescu

et al. 2005a).

in Figure 103. Anisotropy in spatial correlation is addressed, with the horizontal scale of fluctuation

exceeding the vertical scale of fluctuation by one order of magnitude. Figure 104a shows the contours of maximum shear strain for a uniform soil deposit with undrained

strength of 100 kPa and for a prescribed normalised vertical displacement at centre of foundation

$\delta/B = 0.1$. In this case the failure mechanism is symmetric and well-defined. The results of the analyses

indicated that different sample realisations of soil properties corresponded to fundamentally different

failure surfaces. Figure 104b shows an example of a sample realisation in which the spatial distribution

of undrained strength is not symmetric with respect to the foundation. Hence, as could be expected,

the configuration at failure, shown in Figure 104c, involves a rotation as well as a vertical settlement.

Figure 104. Selected results of investigation on homogeneous and spatially random foundation soil (Popescu et al.

2005a).

The repeated finite-element analysis allows appreciation of compound kinematics (settlements and

rotations) of the footings, which could not be inferred from deterministic bearing capacity calcula

tions (i.e. neglecting spatial variability). In general, it was observed that failure surfaces are not mere

variations around the deterministic failure surface; thus, no “average” failure mechanisms could be iden

tified. Another relevant result was the observed significant reduction in the values of bearing capacity

spatially in the heterogeneous case in comparison with the deterministic model: Figure 104d shows that

the normalised pressure required to induce a given of normalised settlement is always higher in the

deterministic case.

Popescu et al. (2005a) also constructed fragility curves accounting for both inherent spatial variability

and epistemic uncertainty in the expected value of soil strength (i.e. induced by measurement, statistical

estimation and model errors). Based on such fragility curves obtained at failure, nominal values of

the bearing capacity of a spatially variable soil deposit corresponding to an exceedance probability of

5% were established for a range of values of probabilistic characteristics of both spatial variability and

expected value of strength.

Fenton & Griffiths (2005) investigated the reliability of shallow foundations against serviceability

limit state failure, in the form of excessive and

differential settlement, both for a single footing and for two footings. Figure 105 shows cross-sections through finite element meshes of: (a) single footing; and (b) two footings. Figure 106 provides a 3-D visualisation of the finite element mesh for the two-footings case. The elastic modulus of the soil was modelled as a lognormal random field with a spatially isotropic correlation structure.

7.5 Seismic liquefaction-induced ground failure

Popescu et al. (2005b), following a number of their own past research efforts, used a Monte Carlo simulation approach involving the generation of non-normal bivariable random fields and non-linear finite element analyses to investigate the effects of soil heterogeneity on the liquefaction potential of a spatially heterogeneous soil deposit subjected to earthquake loading. Both 2D and 3D cases were addressed.

Soil heterogeneity was described using two properties: overburden stress-normalised cone tip resistance and the CPT-based soil behaviour classification index. Gamma and symmetric beta probability distributions were selected for the two parameters, respectively. A negative correlation $\rho = -0.58$ was

Figure 105. Cross-sections through finite element meshes of: (a) single footing; and (b) two footings founded on a spatially heterogeneous soil (Fenton & Griffiths 2005).

Figure 106. 3-D visualisation of the finite element mesh of spatially heterogeneous soil volume supporting two footings (Fenton & Griffiths 2005).

assumed between the two based on the results of previous studies. A separable exponential autocorrelation

model was assigned to both variables. Only the saturated sand was modelled as a spatially random

medium; the dry sand was modelled as deterministic. The calculations were performed for a range of

seismic acceleration intensities. The finite element meshes for the two cases are shown in Figure 107a and Figure 107b, respectively.

The dimensions of the analysis domain were selected as 4-5 times larger than the vertical and horizontal

correlation distances, given by the respective scales of fluctuation (2 m and 10 m, respectively). Given

the small dimensions of the finite elements in comparison with the correlation distances, the variance

reduction effects due to spatial averaging over the elements were neglected. Finite element analyses were performed in two phases. In a first phase, initial effective stresses were

computed by applying gravity loads and allowing full consolidation. Subsequently, nodal displacements,

velocities and accelerations are zeroed and seismic input was applied at the base of the mesh. Base input

accelerations were generated using a procedure by Deodatis (1996) for non-stationary random processes

capable of simulating seismic accelerograms that are compatible with prescribed response spectra and

having a prescribed modulating function for amplitude variation. A different input acceleration time (a) (b)

Figure 107. Finite element meshes for: (a) 3D analysis; and (b) 2D analysis (modified after Popescu et al. 2005).

Figure 108. Comparison between computed contours of excess pore water pressure ratio with respect to the initial

vertical stress for one realisation used in Monte Carlo simulation (Popescu et al. 2005b).

history is generated for each simulation. Each accelerogram

was scaled according to its Arias intensity,

which is a measure of the total energy delivered per unit mass (Arias 1970).

Figure 108 (Popescu et al. 2005b) provides a comparison between the computed contours of excess

pore water pressure ratio with respect to the initial vertical stress for one realisation used in Monte

Carlo simulation for: (a) 3D analysis (results are shown at the midsection shown in Figure 107); (b) 2D

plane strain analysis; (c) deterministic model. Figure 108d shows the contours of normalised cone tip

resistance corresponding to the midsection of the 3D analysis domain and to the 2D analysis domain for

the same realisation.

Figure 109 illustrates the contours of excess pore water pressure ratio on the maximum-effect plane

for the realisation shown in Figure 108 for 8, 12 and 16 seconds of seismic loading. Figure 109d shows

the contours of normalised cone tip resistance on the maximum-effect plane for the same realisation.

The white circle in (d) indicates the initial pore pressure build-up area.

Figure 110 (Popescu et al. 2005b) provides comparative diagrams of smoothed results from deter

ministic, 2D and 3D analyses. The plots are based on 300 Monte Carlo simulations (for both 2D and

Figure 109. Contours of excess pore water pressure ratio on the maximum-effect plane for the realisation shown in

Figure 108 at time: (a) 8 seconds; (b) 12 seconds; and (c) 16 seconds of the acceleration time history. Plot (d) shows

the contours of normalised cone tip resistance on the maximum-effect plane for the same realisation (Popescu et al.

2005b).

3D) and corresponding deterministic analyses. The results in Figure 110 are smoothed out using a mov

ing average window technique. Popescu et al. (2005b) noted the substantial agreement between the

results of 2D (Figure 110a) and 3D (Figure 110b) analyses for excess pore water pressure ratios (both

volumetric and maximum-effect plane average values). For such parameters, the deterministic analyses

yielded unconservative results for low-intensity inputs and slightly greater values for higher-intensity

loading. No significant differences were observed in terms of horizontal displacements (Figure 110c)

while it may be seen in Figure 110d that spatial heterogeneity and 3D effects are both relevant factors in

modelling the maximum liquefaction-induced ground settlement. Results suggest that a 2D model may

be unconservative, and even more so the deterministic model in which spatial variability is neglected.

Assimaki (2006) reported that spatial variation is important in seismic response analyses when the cor

relation distance is comparable to the wavelength of the incident motion and high frequency predictions

are affected by complex scattering effects. The results of the analyses were also presented in the form of fragility curves expressing the probability

of exceeding a certain response threshold as a function of earthquake intensity. Figure 111 compares the

results of stochastic (2D and 3D) and deterministic analysis in terms of fragility curves. The qualitative

differences described for Figure 110 may be identified in Figure 111 also.

7.6 Sliding of retaining walls

Fenton et al. (2005) investigated the failure behaviour and the reliability of a two-dimensional frictionless

wall retaining a cohesionless drained backfill. Soil friction angle and unit weight are modelled as spatially

variable properties using lognormal random fields with single exponential-type correlation structures. Figure 112 shows the active earth displacements for two realisations of the finite element mesh. The

location and shape of the failure surface is strongly related to the presence of weaker soil zones (shown (b) (d) (a) (c))

Figure 110. Comparative diagrams of smoothed results of deterministic, 2D and 3D analyses: (a) maximum

value of the volumetric average of excess pore pressure ratio; (b) maximum value of excess pore pressure ratio on

maximum-effect plane; (c) maximum predicted horizontal displacement at ground level; and (d) maximum predicted

settlement at ground level (Popescu et al. 2005). (a) (c) (b) (d)

Figure 111. Stochastic (2D and 3D) and deterministic analysis in terms of fragility curves: (a) probability that maximum

value of volumetric average of excess pore pressure ratio exceeds 0.6; (b) probability that maximum of maximum-effect

plane average of excess pore pressure ratio exceeds 0.7; (c) probability that maximum horizontal displacement exceeds

50 mm; (d) probability that maximum settlement exceeds 25 mm (Popescu et al. 2005b). (a) (b)

Figure 112. Active earth displacements for two realisations (both having the same correlation distance and coefficient

of variation) of the random field of soil friction angle (Fenton et al. 2005).

in lighter colours in the finite-element mesh) and is, in both cases, markedly different from the simple

shapes which are assumed in earth pressure theory (e.g. planar in the Rankine model). In the study by Fenton et al. (2005), unit weight and friction angle random were first assumed to be

uncorrelated, then strongly correlated. An analysis of the results indicated that the correlated case yielded

lower probabilities of failure than the uncorrelated case, which can thus be regarded as conservative.

Again, a critical value of the scale of fluctuation, for which active loading on the retaining wall was

maximised, was observed.

8 CLOSING STATEMENTS

Newer design codes recognise and address the uncertainties in soil properties and engineering models.

Soil variability then assumes an increasingly important role in practice and research. This should be seen

as a positive evolution of the geotechnical discipline, as addressing uncertainty in a consistent manner

allows more economic and rational design. The paper provides an overview of selected techniques for modelling the inherent variability of soils.

A perspective as practical as possible was pursued, with wide reference to available literature. Examples

from probabilistic slope stability analyses were illustrated to highlight the benefits and limitations of

approaches with various levels of complexity. Updated data tables were provided for illustration purposes.

Most statistics available in the literature are strongly siteand case-specific, and the data should be

examined with caution if they are to be applied at other sites. Research may help simplify the use of variability-modelling techniques, thus assisting the practitioner.

However, even the most powerful modelling technique can yield unreliable results if input data are

insufficient in quantity and quality. Geotechnical practice makes use of data sets which invariably

indicate variability in any soil property. The variability information is often lost in the characterisation

and design processes. A first step towards an uncertainty-based approach in geotechnical practice could

be the explicit reporting of properly obtained data statistics. At present, research efforts focus on a variety of aspects of soil variability modelling. Advanced

simulation techniques, for instance, allow bypassing some of the barriers encountered in past efforts.

Enhanced capabilities of computing tools and use of sophisticated integrated methodologies make it

possible to model with increasing realism the behaviour of complex geotechnical systems. Geotechnical

practice, on the contrary, still largely relies on deterministic approaches. The gap between geotechnical

research and practice needs to be narrowed. It is perhaps illusory that such gap may ever be eliminated. However, a greater acceptance of variability

as a major actor in geotechnical practice would help reduce such gap. Reduction of the gap should occur

from both sides: research should merge the mathematical techniques and geotechnical data into a more

readily usable format; practice should increasingly recognise the importance of addressing data in the

light of uncertainty, and accept the necessity to acquire additional competence regarding the statistical

treatment of data.

There is little doubt that a shift to an uncertainty-based perspective is taking place. The joint effort of

researchers and practitioners should aim towards a full recognition of the benefits of such development.

ACKNOWLEDGEMENTS

The authors wish to acknowledge the collaboration of Jostein Jerkø (on internship at NGI from NTNU)

in reviewing the literature and compiling the data tables, as well as the assistance by Tini van der Harst

from NGI in formatting the paper.

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Engineering characteristics of Taipei Clay

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ABSTRACT: Taipei City is located within the Taipei Basin on the northern part of Taiwan. The cohesive

deposit of the soils is referred as Taipei Clay, which is the soil deposit of most significant engineering

concern. Due to rapid development of the city, major building and infrastructure projects demand a

comprehensive understanding of the soil deposits. The engineering characteristics of Taipei Clay have

been extensively investigated in those major projects. In this paper, the geology of Taipei City will be

reviewed first. This section is followed by the results of geotechnical zoning of Taipei City. It provides

an overall picture of the extent and variability of soil deposits. Then, detailed studies of Taipei Clay are

presented, including state of the clay, advanced laboratory test results, and in situ test results. Finally,

significant engineering problems encountered by the geotechnical engineers are discussed.

1 BACKGROUND

Taipei City is the most populated city in Taiwan. It is located in the northern part of Taiwan Island and

occupies more than half of the area of the Taipei Basin (Figure 1). Due to rapid economic development

of Taiwan over the past several decades, many high-rise buildings and major infrastructures have been

built in Taipei. All these constructions have been carried out in the recent Quaternary deposits within

the Taipei Basin.

Before 1960s, construction within the Taipei Basin was carried out with minimum knowledge of the

soils. Most studies on the deposits are from geologic point of view. The first systematic engineering study

was conducted by Hung (1966). He named the whole Sungshan

Formation, the Quaternary deposits of

the Taipei Basin, as the "Taipei Silt". He also identified six interbedded silty clay and silty sand layers

for the Quaternary deposits. Results of a clay mineralogy study and a summary of soil index properties

were reported in his report. Groundwater conditions have also been briefly discussed in that study.

More than a decade after Hung's study, Moh & Ou (1979) conducted a more comprehensive study

on Sungshan Formation with better sampling techniques and more sophisticated laboratory equipments.

The properties of six alternating layers of silty clay and sand of Sungshan Formation were further

investigated. The particular interest of the paper is the emphasis of silt content. The "six-layer model"

introduced by Hung (1966) has been adopted in this paper. Moh & Ou (1979) still used "Taipei Silt" to

describe the whole Sungshan Formation: "The subsoil stratum which is of prime concern in this paper

is the Taipei Silt. In general, the uniformly distributed soil stratum is composed of alternative layers of

low plasticity silty clay (ML-CL or ML) and silts (ML), interstratified with fine sand layers containing

high silt content. The silt content in the various layers ranges from 15% to 95%". Moh & Ou (1979)

is the first paper, from an engineering point of view, to highlight the impact of deep well pumping and

its associated ground subsidence on the construction of foundations and other underground structures.

It should be noted that piezometric pressures measured at that time (1979) are much lower than the

assumed hydrostatic pressure distribution due to significant deep well pumping.

In early years, most studies have been carried out around the Taipei Main Station where is the downtown

area of Taipei at that time. However, with more projects in different places of the Taipei Basin, it is found

that the thickness of each stratum varies and properties of the soils also vary. The "six-layer model" is still

applicable, but it needs refinement to better delineate the engineering properties of soils in the Basin. The

first city-wide geotechnical mapping was conducted and completed by Moh and Associates, Inc. (MAA)

in mid-1980's (MAA 1987). This study divided the city area into six geotechnical zones based on a very

scrupulous review of physical properties of soil samples and the depositional environment of three rivers

Figure 1. Satellite Image of the Taipei Basin (Copyright (c) CSRSR/CNES, 2004, Courtesy CSRSR, NCU).

flowing into the Basin Figure 2. This study had great implications for further researches as well as for engi

neering practice. Several geotechnical zoning and microzoning works have been conducted on the basis of

this study (e.g. Cheng 1987, Lee 1996). Results of this study have been used in the planning of major engi

neering projects in Taipei City, such as the Taipei Rapid Transit Systems (TRTS). Since it further validated

the "six-layer model", geotechnical engineers in Taipei can better describe the soil layer they refer to, say,

Layer II of K1 Zone. In addition, a series of engineering correlations have been established (MAA 1987). As the TRTS got started, there were much stronger needs to understand behaviors of the cohesive

soils in the Taipei Basin. A series of studies have been carried out (Chin et al. 1989, Liu et al. 1991,

Chin & Liu 1997) to evaluate whether the silty clay in Taipei possesses the normalized behavior and

how significant is the anisotropy of the undrained shear strength. Test results indicate that the cohesive

deposits in Taipei behave just as a low plastic clay, which implies that the cohesive deposits in Taipei

can be generally modeled as other clay deposits. One of the major endeavors in the study of the soil

properties was the Designated Task of TRTS. It was carried out by MAA, the Geotechnical Engineering

Specialist Consultant (GESC) of TRTS, with the participation of MIT and Golder Associates. Some

high quality samples have been tested at MIT. These test results can be treated as a set of “benchmark”

test results for further studies of Taipei soils. Later on, soil parameters of the MIT-E3 model have been

derived from these results and used in an advanced numerical analysis of deep excavations.

Figure 2. Geotechnical zoning of the Taipei Basin and the priority network of TRTS.

After the completion of the initial TRTS network, construction in Taipei City has continued to evolve

into a new era that demands high-quality and advanced sampling, testing, and modeling technology for

much more difficult construction environments.

This paper presents a comprehensive overview of behaviors and properties of Taipei Clay collected

from numerous field and laboratory experiences. Lessons learned from the past and information sum

marized in this paper can provide better insight and guidelines for future construction in Taipei City. It

has to be noted that “Taipei Clay” is the cohesive deposits of Sungshan Formation in the Taipei Basin.

Furthermore, Layer IV of the Sungshan Formation often occurs at about the invert of open excavations

and is the medium through which tunnels will pass for the TRTS. Hence, Layer IV clay has been the

focus of past studies, which rendered significant data for this paper. For Layer IV materials in K1 Zone,

liquid limit (w_l) is about 35% and plasticity index (I_p) is about 13% (MAA 1987). It is interesting to

note that these index properties are not much different from those of Boston Blue Clay. For Boston Blue

Clay, the liquid limit is about 42% and the plasticity index is about 19%. Hence, when possible, relevant

engineering properties of these two clays will be compared. In this paper, the geology of Taipei City will be reviewed first. This section is followed by the results

of geotechnical zoning and information on stratigraphy of the city. It provides an overall picture of

the extent and variability of soil deposits in the Taipei Basin. Statistical characterization of physical

properties for soils in each zone concluded from the mapping study also will be given. Then, detailed

studies of Taipei Clay are presented, including state of the clay, advanced laboratory test results, and in

situ test results. Finally, significant engineering problems encountered by the geotechnical engineers are

discussed.

2 GEOLOGY OF TAIPEI CITY AREA

2.1 Geology of the Taipei Basin

2.1.1 Overview

The Taipei Basin was formed as a result of reverse and normal faulting in northern Taiwan. The reverse

faults, including the Hsinchuang Fault on the northwest, the Kanchiao Fault on the northeast, and the

Taipei and Hsintien Faults on the southeast of the Taipei Basin, are generally trending NE or ENE with

thrust upward from southeast to northwest. The most significant normal fault, Shanchiao Fault on the

west of the Taipei Basin, is trending NWN with a dip toward ENE. The geology surrounding the basin

can be distinguished into three geological zones Figure 3, the Tatun Volcanic Group to the north, the

Tertiary Sedimentary rocks to the southeast, and the Quaternary Linkou Tableland to the west (Wang

Lee et al. 1978, Teng et al. 1994). The basin is filled with "unconsolidated" Quaternary and Recent sediments Figure 4. Most materials

deposited in the basin have been derived mainly from the weathering products of the peripheral Tertiary

formations through the three major rivers, the Tahan River, the Hsintien River, and the Keelung River.

They were also partly from the eruption and weathering products of the Pliocene to Pleistocene andesite

and basalt of the Tatun Volcano Group and the Kuanyinshan Volcano. Taipei Clay is referred to the

cohesive deposits of the Quaternary sediments in this paper.

2.1.2 Depositional environment

The depositional process and environments of the Quaternary sediments in the Taipei Basin (Table 1)

are summarized in ascending order as follows (Chou 2005):

(a) Panchiao Formation: The basal gravel beds, sandy and muddy sediments of the Pleistocene Panchiao Formation in the Taipei Basin were deposited under the alluvial fan, fluvial and lacustrine environments. The thickness of the Panchiao Formation ranges from 0 to 397 m and varies with basal topography of the Miocene basement in the Taipei Basin. The Panchiao Formation is the thickest in the Wuku area of the western part of the Taipei Basin, and it gradually thins out toward the northeast and the southeast in the Basin.

(b) Wuku Formation: The gravelly, sandy and muddy sediments

of the late Pleistocene Wuku Formation is also the thickest in the Wuku area, being around 142 m in thickness. It gradually thins out toward the northeast and the southeast in the Basin.

(c) Chingmei Formation: The yellow quartzite gravel bed of the late Pleistocene Chingmei Formation, in the Taipei Basin, were deposited entirely under the alluvial fan environment. The thickness of the Chingmei Formation ranges from 30 to 138 m. The greatest thickness of Chingmei Formation is in the Chingmei area, and it gradually thins out toward the north.

(d) Sungshan Formation: The sandy and muddy sediments of the late Pleistocene to Holocene Sungshan Formation in the Taipei Basin have been deposited under the fluvial, lacustrine, estuary and brackish water bay environments. The thickness of the Sungshan Formation ranges from about 20 to 120 m. The greatest thickness of Sungshan Formation is in the Wuku area, and it gradually thins toward the southeast and the northeast. Near the end of the deposition of the formation, the brackish water gradually retreated from the basin. Numerous plants grew in the marsh and were later buried to form the peat beds.

Figure 3. General geology and well locations of the Taipei Basin (adapted from Wang Lee et al. 1978,

Teng et al. 1994).

Figure 4. Stratigraphic column of the Miocene and Quaternary in the Taipei Basin (formation data from Wang Lee

et al. 1978, Teng et al. 1994, Wu 1965, Liu et al. 1994, Wei et al. 1994, Chen et al. 1995, Wang et al. 1996, Su

et al. 1999).

2.1.3 Rate of sedimentation

According to the radiocarbon dating results (Liu et al. 1994), rates of the sedimentation of the late

Pleistocene to Holocene in Sungshan Formation are estimated to be from 1.10 to 6.03 mm/year, and the

greatest rate happened in the Wuku area to the west of Taipei. The sedimentation rates of the late Pleistocene Chingmei Formation are estimated to be from 0.80 to

1.60 mm/year from results of radiocarbon dating made by Liu et al. (1994). The rate is slightly higher

Table 1. Depositional environment and sedimentation rate of Quaternary sediments in Taipei Basin (Chou 2006).

Sedimentation rate (mm/year)	Formation	Depositional environment
0.02-1.98	Hsinchung	Alluvial fan-fluvial-lacustrine
1.58-3.54	Wuku	Alluvial fan-fluvial-lacustrine
0.80-1.60	Chingmei	Alluvial fan-fluvial-lacustrine
1.10-6.03	Sungshan	Fluvial-estuary-lacustrine-brackish water bay

in the Chingmei area of southern Taipei Basin. The sedimentation rates of the late Pleistocene Wuku

Formation are estimated to be from 1.58 to 3.54 mm/year based on results of radiocarbon dating made

by the researchers (Liu et al. 1994, Wang et al. 1996). It is slightly higher in the Wuku area of western

Taipei Basin.

The sedimentation rates of the late Pleistocene Panchiao Formation are estimated to be from 0.02 to

1.98 mm/year based on the results of radiocarbon and $^{40}\text{Ar}/^{39}\text{Ar}$ dating results (Wei et al. 1994, Wang

et al. 1996). The greatest rate is in the Wuku area and gradually reduces from there toward the east and

the south of the Taipei Basin.

The sedimentation rates of Quaternary sediments in the Taipei Basin are also summarized in Table

1. Generally speaking, the sedimentation rate of the Quaternary sediments in the Taipei Basin is slower

since 0.4 Ma and gradually increases toward the Holocene.

2.1.4 Water depth

The deposition of the late Pleistocene to Holocene Sungshan Formation in the Basin was under the

fluvial, estuary, lacustrine and brackish water bay environments characterized by shallow depth, weak

current and quiet water. These conditions are verified by the texture and fossil content of the sediments.

The Recent water level of the Tamshui River and the Keelung River ranges from 0.1 m to 1.3 m during

the high tide and from -0.4 m to 0.06 m during the low tide.

2.1.5 Stress history

Since the Taipei region subsided around 0.3-0.4 Ma and the Taipei Basin formed, the area has undergone

a continuous sedimentation process. Hence a new land, the Taipei Basin, has been developed under the

alluvial, fluvial, lacustrine and brackish water sedimentation and probably without recognizable erosion.

2.1.6 Tectonics

Due to the oblique collision between the Eurasian Plate and Philippine Sea Plate since the Pliocene to

Pleistocene, the Tertiary sequences have been folded and faulted, and the northeast trending Chinshan

Hsinchuang Fault, the Kanchiao Fault, the Taipei Fault, the Hsintien Fault and the Chengtzuiliao Fault

were formed in Taipei area successively (Figure 3). And then the Shanchiao Fault, the Neihu Fault, the

Tachih fault, the Kungkuan Fault and the Taita Fault were formed due to the great subsidence and normal

faulting of the Taipei area since around 0.4 Ma (Chou 2005).

The eruptions of the Tatun and Kuanyinshan Volcanoes caused the great subsidence of the Taipei

region. The Shanchiao Normal Fault has been successively formed parallel with the Hsinchuang Fault

along the northwestern border of the Taipei region since around 0.3-0.4 Ma, and the Taipei Basin has

been formed. The Shanchiao Fault is still active based on

the seismic exploration and the observations

on the subsidence of the Taipei Basin (Wang et al. 1994, Wang et al.1995, Wang et al. 1996, Yu et al.

1994, Chou 2005).

Due to gradual compaction of the thick and loose late Quaternary sediments as a result of sediments

loadings and shakings by the successive earthquakes, the southeastern hanging wall of the Shanchiao

Fault gradually subsided to form the Wuku gravity normal growth fault. These faults offset the late

Quaternary loose formations for about tens of meters nearly vertically, and they are still moving and

active (Wang et al. 1996, Wang et al. 1999, Yu et al. 1994, Chou 2005).

2.1.7 Temperature

The pollen analysis of the Wuku no.13 core in the western Taipei Basin provided a detailed record of the

paleoclimatic changes in the Taipei Basin (Tseng & Liew 1999). The forest-grassland pollen assemblage

demonstrated that the cold-dry period of the last glacial changed at about 17100 yr. BP (79.8 m in depth)

when arboreal pollen elements increased. The shallow water environment appeared in the Basin because

of the rise of the aquatics since 12400 yr. BP (69.2 m in depth). The climate trended into warm-wet at

about 10800 yr. BP (65 m in depth) due to dominance of Fagaceae. An episodic cool phase in about

10200 yr. BP (63.4 m in depth) was probably correlated to Younger Dryas cooling event when the cool

temperature elements increased, then yellow erosive soil appeared and water level lowered. Obviously,

increased tropic-subtropical and warm-wet arboreal pollen elements, the coarser sediments, and the

highest sedimentation rate indicated the warm-wet climate since 9100 yr. BP (50.4 m in depth). The

abnormal at about 7300 yr. BP (39.8-23.4 m in depth) may due to the intensification of monsoon in

the warm-wet climate. There were created clearances prior to the changes of vegetation probably due to

the human activity at about 3800 and 3500 yr. BP (10.65 and 10.1 m in depth). According to the pollen

assemblage of the palynological study by Shaw et al. 1999, the upper part of Sungshan Formation is

considered to be the most hot and humid stage in the Taipei Basin.

2.1.8 Bioturbation

The foraminifer, ostracod and mollusk fossils are found in almost every member or lentil of Sungshan

Formation in the Taipei Basin. The foraminiferal fauna in Sungshan Formation is composed largely of

benthonic forms with subordinate pelagic species. The Taipei Basin foraminiferal fauna is characterized

by the predominance of benthonic species, especially two genera, *Streblus* and *Elphidium*. The abundant

benthonic components and the extremely scarce pelagic species imply that the depositional environ

ment of the sediments is mainly shallow water. Two leading species, *Streblus annectens* and *Elphidium*

tainanensis are characteristics of the brackish water form (Huang 1962). The mollusks, *Cotbicula formosana*, *Ostrea gigas*, *Placuna placenta*, and *Sinonovacula constricta*

were found in Sungshan Formation. *Cotbicula formosana* was found by Tan (1939) in the present estuary

of the Tamshui River (Kuroda 1941). The other fossils found in Sungshan Formation consist of Balanids, Ostracods, fish bone, and plant

remains (Huang 1962). Therefore, Sungshan Formation is weakly bioturbated based on the containing

fossils.

2.1.9 Local geological features: engineer's concerns

In the Taipei Basin, methane and drift woods are the two unique geological features (Moh & Hwang

1997). Methane was encountered in several boreholes during investigation. Its presence is related to the

capture of gas in dooms encapped by impermeable clays (Lin et al. 1997). In one case, during the drilling

of a borehole of the Chungho Line of TRTS, the eruption sent a jet of methane-water mix to a maximum

height of about 10 m into the air and continued for more than 3 days. Contractors have been warned for

the possibility of encountering methane in tunnel drives and possible consequences. As a precaution,

specifications require the concentration of methane be continuously monitored during tunneling of TRTS

and shield machines be equipped with detective devices and alarm systems. Gas problem has also been

encountered during some basement excavations in Taipei (Liu 2006). It should be noted that methane

could be encountered at locations all over the Taipei Basin. Drift woods were recovered in numerous excavations and were responsible for a few accidents encoun

tered during tunneling (Lin et al. 1997, Ju et al. 1997). Chunks of 1 m or so in diameter were frequently

encountered and they may be as long as 5 m. A piece of wood found at a depth of 9 m in Observation

Well 2 (W-1797) in Panchiao dated back to 6760 years and that found at a depth of 23 m dated back to

7,950 years (Liew 1994). The presence of drift woods was well recognized and has been well reported,

however, it is still very difficult to predict the exact locations and depths of drift woods beforehand. The

drift wood problems could be very serious for shield tunnelings. All the shield machines adopted for

TRTS have sufficient strength and power to cut through the woods as long as they are not too large in size.

However, there was one occasion in which the presence of drift wood caused the ground to collapse as

the movement of the shield was hampered and soft clay kept on moving into the earth chamber. Finally,

a hole of 5 m in diameter occurred right above the head of the shield machine. Workers entered the earth

chamber and recovered two pieces of drift wood which were 500 mm and 400 mm in length, respectively

(Moh & Huang 1997).

2.2 Composition

2.2.1 Form of clay fraction

The clay fractions of Sungshan Formation in the Taipei Basin are mainly composed of non-clay minerals,

clay minerals, and a small amount of biogenic debris. Most of the non-clay minerals are concentrated

largely in the silt-size and sand-size fractions. The non-clay minerals are composed mainly of quartz, coal,

and rock fragments with a small amount of feldspar, muscovite, biotite, limonite, vivianite, hornblende,

pyroxene, and undetermined minerals (Huang 1962, Chou 2005). The minerals of the clay-size fraction

finer than 2 microns of Sungshan Formation were studied by X-ray diffraction. The clay minerals are

mainly composed of illite with some chlorite. The rock fragments are mainly of fragments of fine

sandstone, quartzose sandstone, slate, shale, and schist (Huang 1962).

The biogenic debris in Sungshan Formation is composed of foraminifers, ostracods, mollusks, bal

anids, coal, peats, pollens, and spores. The foraminiferal fauna in Sungshan Formation is composed

largely of benthonic forms with subordinate pelagic species. The Taipei Basin foraminiferal fauna

is characterized by the predominance of benthonic species, especially two leading species, *Streblus*

annectens and *Elphidium taiwanensis* (Huang 1962). The ostracods are mainly of *Cytheris* sp. In the

present assemblage of the mollusks, *Corbicula formosana*, *Dentalium* cf., *robustum*, *Dosinia grunerii*,

Natica maculosa, *Nassarius hepaticus*, *Ostrea gigas*, *Ostra* spp., *Placuna placenta*, *Pyrene bella*, and

Sinonovacula constricta are found in Sungshan Formation. The fossil *Balanus* aff. *amphirata* was found

in association with mollusk fossils.

2.2.2 Organic content and form

The peat beds were found in the upper part of Sungshan Formation in the southern Taipei Basin. The coal

fragments derived probably from the Miocene coal-bearing beds were found in Sungshan Formation.

2.2.3 Mineralogy

The clay-sized fraction of the late Quaternary sediments in the Taipei Basin is mainly composed of

clay minerals with minor amount of non-clay minerals, which are usually quartz, feldspars, limonite

(mainly goethite), gibbsite, and calcite etc. The clay minerals found in the sediments are illite, chlorite,

kaolinite, montmorillonite, beermiculite, and mixed-layer clay minerals. As a whole, the illite, chlorite,

and mixed-layer clay minerals are three major clay minerals

in the sediments. On the contrary, kaolinite

and montmorillonite are the minor components. Illite and chlorite have an obviously increasing trend of

abundance, from the Lower Member of Hsinchuang Formation (Panchaio Formation) up to Sungshan

Formation, while kaolinite has an obviously decreasing trend. Montmorillonite and mixed-layer clay

minerals roughly have a decreasing tendency upwards. This phenomenon is closely related to the gradual

change from the mainly alluvial depositional environment at the earlier stage to the brackish lacustrine

environment at the later stage of the sedimentation of the Taipei Basin (Lin & Chen 1999).

During the subsequent sedimentation through the Upper Member of Hsinchuang Formation (Wuku

Formation), Chingmei Formation up to Sungshan Formation, more and more sedimentary materials were

derived from the Tertiary sedimentary province and the very low-grade metamorphic province in the

southeastern, both of which are further from the Taipei Basin. Both the kaolinite content in Sungshan

Formation and the montmorillonite content in Chingmei Formation gradually increase from the southern

part to the northern part of the Taipei Basin, implying that more and more sediments were supplied from

the northern volcanic region and the northwestern Linkou Tableland (Lin & Chen 1999).

In addition to these geological reviews, a more recent study was carried out at National Taiwan Uni

versity of Science and Technology (Chin & Liu 1997). To understand the mineralogical properties of

Taipei Clay, the X-ray diffraction and scanning electron microscope (SEM) were carried out. The sam

ples used for the investigation were taken from Layer IV of K1 Zone. Figure 5 shows a typical result of

Figure 5. X-ray diffraction of clay at R-1 site (after Chin & Liu 1997). (a) Horizontal view (b) Vertical view

Figure 6. Microfabric of Taipei Clay viewing using SEM (after Chin & Liu 1997).

the clay, which suggests that the predominant clay mineral is illite (78%) and kaolinite (20%). Previous

studies (e.g. Hung 1966) suggested that predominant clay minerals for the cohesive Sungshan Formation

materials are illite and chlorite. The illite dominance in Taipei Clay is obvious. Microfabric of this K1

clay is shown in Figures 6a and 6b by the horizontal and vertical SEM results, respectively. The stack of

illite plates can be more clearly seen in the vertical direction than in the horizontal view. In the horizontal

direction, the arrangement of illite plates is somewhat random. For samples tested in this program, the

natural water content is about 38%, the liquid limit and plasticity index are approximately 40% and 18%,

and the specific gravity is about 2.73.

2.3 Groundwater and ground subsidence in the Taipei Basin

2.3.1 Hydrology of the Taipei Basin

Records of core logs and groundwater levels in the Taipei Basin indicated that the sediments between the

ground surface and Chingmei Formation or the gravel materials at the bottom of Sungshan Formation

constitute a regional confining bed. The confining bed consists mostly of silty and clayey materials, but

it also contains several sand layers. The hydraulic head of these local sandy aquifers generally decreases
55 50 45
40 35 30 25 20 15 10 5 0 1960 1965 1970 1975 1980
1985 1990 1995 2000 2005 Year Piezometric level (m)
0.00 0.25 0.50 0.75 1.00 1.25 1.50 1.75 2.00 225 S

element (m) Data before 1997 from China (1997)
Piezometer level Settlement

Figure 7. Groundwater drawdown and ground settlement in Taipei.

with the depth. Hsinchuang Formation, Chingmei Formation, and the gravel materials at the bottom of

Sungshan Formation make up a regional aquifer. The aquifer is primarily composed of highly permeable

gravel and sand bed. The major recharge areas are the river valleys of the Tahan River and the Hsintien

River. A minor amount of recharge, however, is supplied by the downward flow through the overlying

confining bed (Chia et al. 1999).

2.3.2 Groundwater pumping and ground subsidence

During the development of the modern city of Taipei, changes in the groundwater regime have been

dominated by the extensive pumping from Chingmei Formation to supply water for the city. Similar

to other cities that have experienced the same phenomenon, significant pumping-induced settlement

occurred.

Figure 7 shows the typical water pressure drawdown in the Chingmei gravel and the associated settle

ment data for the downtown area (i.e. Taipei Main Station) of Taipei. Pumping of the groundwater from

the Chingmei gravel began prior to 1960. Despite of restrictions imposed by the government in 1968,

pumping from the Chingmei Formation continued until the mid-1970s, when the greatest drawdown

was reached. Thereafter, there is a trend of recovery of water pressures in the gravel layer indicating the

reduction in pumping. As shown in Figure 7, the ground surface settlement associated with pumping is

closely correlated with the piezometric level within Chingmei Formation.

Another effect of water pressure reduction in the Chingmei gravel was the reduction in water pressures

in the overlying Sungshan deposits. The effect of pressure reduction extended through the bottom three

layers of Sungshan Formation (i.e. Layers I to III. Layer IV), which consists of silty clay, acted as an

aquitard and essentially preserved hydrostatic conditions in the overlying materials. Figure 8 focuses on

these changes of groundwater pressure profile within Sungshan Formation over the past 30 years. As

shown in Figure 8, in 1980, prior to the TRTS construction, the groundwater pressure in the lower 4

layers of Sungshan Formation is significantly lower than the hydrostatic pressure. Measurements in late

1990's have shown consistent increases in the piezometric pressure that approaches toward the hydrostatic

condition.

2.3.3 Groundwater recovery

As shown in Figures 7 and 8, the water pressures in the Chingmei gravel and the lower Sungshan deposits

have been recovering since the mid-1970's. Figure 9 illustrates this groundwater recovery from 1974

to 1995 by including the typical water pressure data in the Chingmei gravel, Layer III of the Sungshan

deposits, as well as measurements from Layer V. The data indicate a relatively slow rate of recovery

Year	Water pressure (kN/m ²)	Elevation (m)	Layer
1980	100	100	Layer VI (CL)
1974	100	100	Layer V (SM)
1980	100	100	Layer III (SM)
1990	100	100	Layer VI (CL)
1986	100	100	Layer I (SM)
2004	100	100	Assumed hydrostatic
1997	100	100	Layer IV (CL)

before 1997 from China (1997) Layer IV (CL)

Figure 8. Groundwater pressure profile in the Taipei Basin.

Year 50 40 30 20 10 0 10 Elevation (m) 1974
 1975 1980 1985 1990 1995 Layer V Layer III Dorts wells at
 Taipei Main Station Layer III (MAA) Chingmei gravels range
 of data for 8 wrpc wells Chingmei gravels wrpc wells at new
 park

Figure 9. Piezometric level of Chingmei gravel in the Taipei Basin (after Chin 1997).

between 1976 and 1981. From 1981 to 1988, the recovery rate increases with an average head of +2 to

3 m/year. The recovery trend of Layer III closely follows the pressure within the Chingmei gravel while

the piezometric head in Layer V is independent from the underlying aquifer. Despite of the consistent

groundwater recovery, recent monitoring records indicate that the current groundwater pressure is still

below the hydrostatic level (Figure 8). It should be noted that the slight drop in groundwater pressure in

1994 is due to large scale TRTS pumping from the Chingmei gravel. The TRTS construction dewatering

reached its peak stage in 1993-1995 and exceeded the rate of natural recharge. The past pumping with large reductions in groundwater pressure in the Taipei Basin have generally

benefited previous excavations in the city. These excavations, typically for building basements, were

well above the piezometric level in the lower Sungshan deposits; therefore groundwater control measures

were not required. The performance of these braced excavations was reasonably good in terms of wall

deflections and ground movements; the favorable groundwater conditions undoubtedly contributed to

the good performance.

For the TRTS excavations, groundwater control measures were required as the combination of greater

excavation depth and the groundwater recovery places the excavation invert well below the piezometric

level. Several construction failures occurred at TRTS sections which can be attributed to the higher

groundwater pressure; therefore, the control of these water pressures is critical in achieving stability of

the walls and limiting wall deflections and ground movements. Nevertheless, the past regional pumping

also has some beneficial effects on the TRTS excavations. The past pumping in the Taipei Basin have

caused the lower Sungshan deposits to become overconsolidated, thereby, reducing the compressibility

of the cohesive layers to 10%-20% of that in the normally consolidated range. Since pumping associated

with TRTS construction would caused less water pressure reduction in compressibility also translates

into limited and acceptable level of surface settlements even with groundwater drawdown outside of the

excavation (Chin 1997).

It should also be noted that the amount and the distribution of groundwater pumping can vary in the

Basin. The monitoring results of this section are mainly from the area around the Taipei Main Station.

Suburban areas, such as Nankang and Chingmei, can have different groundwater pressure distributions

(Chin et al. 1991).

3 GEOTECHNICAL ZONING AND STRATIGRAPHY

3.1 Geotechnical zoning

In mid-1980's, geotechnical engineering mapping of Taipei City has been carried out. It is aimed at

dividing the basin area of Taipei City into zones according to the depositional characteristics, stratifica

tions and soil properties (MAA 1987). Huang et al. (1987) presented a summary on the methodology of

this study and presents the results of the zoning.

In this study information on representative boring logs was collected from major projects within the

city. More than seven hundred boring logs were compiled. Results of laboratory tests on samples taken

from these boreholes were also reviewed and then included in the database. The zoning was achieved by

the use of numerous fence diagrams, soil profiles, isopachous maps and isohyps maps of each sublayer

and the surface of the bearing stratum. The isopachous map gives the isopachs which are lines of equal

thickness of each sublayer. The isopachous maps provide a very clear idea about the variation of the

thickness of each sublayer. The isohyps maps show the isohypsies which are lines of equal altitude of the

top of each layer. Since the elevation of the top of each sublayer can be obtained from corresponding

isohyps maps, the drawing of geological cross section can be made fairly easily. With all these plots, the

transformation of subsoils can be realized. The zoning was then made under the following two guidelines:

1. The drainage basins of the Tamshui River, the Keelung River, and the Hsintien River are the main

regions of zoning.

2. The zones in each specific drainage basin are further divided according to distribution of the sublayers

and the depositional environment.

The geotechnical zones were preliminarily delineated following these guidelines. Statistical analysis

of the physical properties and engineering properties of the soils in each zone were then carried out. The

zoning and statistical study process were iterated until a

reasonable agreement was achieved.

Figure 2 shows the result of geotechnical zoning. Sungshan Formation can generally be classified into

three major zones based on the drainage basin. Due to differences in location or depositional environment

within the same drainage basin, each major zone can be further divided into 2 to 3 subzones. The

relationship between the various zones and the main drainage basins is shown in Table 2.

The extent and characteristics of the strata in Zone T1, Zone T2, and Zone T3 are briefly described

below:

(1) Zone T1 - Zone T1 is the area located between the Tamshui River and the Hsintien River. Due to

possible influence of shift of the river course, the variation between the layers is very complex. Layer

VI is not distinct in most part of this zone and Layer V is about 15 m thick. Descriptive statistics of

physical properties of T1 soils is given in Table 3.

(2) Zone T2 - Zone T2 is the old downtown area of Taipei City, where is the area around the Taipei Main

Station. The stratification characteristics of the six layers of Sungshan Formation in this zone are most Table 2. Division of geotechnical zoning of the Taipei Basin. Drainage basin Geotechnical engineering zone Tamshui river T1, T2, T3 Keelung river K1, K2 Hsintien river H1, H2

Table 3. Descriptive statistics of physical properties of T1 soils (after MAA 1987). Grain Size distribution (%) Statistical Thickness γ d W n

Layer items (m) (kN/m³) (%) G s w l (%) I p (%) Gravel Sand Silt Clay

VI mean 3.7 16.8 26.6 2.68 33.3 11.2 - 34 31 26 COV 0.27
0.11 0.24 0.02 0.17 0.19 - 0.53 0.25 0.17 n 7 10 10 8 10 9
10 10 7 7

V mean 11.6 18.5 18.7 2.71 - NP 11 63 16 5 COV 0.25 0.12
 0.38 0.004 - - 1.49 0.21 0.51 1.14 n 9 44 44 35 - - 45 20
 20 20

IV mean 15.1 17.0 30.6 2.72 26.9 8.3 1 22 43 26 COV 0.43
 0.14 0.17 0.01 0.21 0.31 6.71 0.67 0.2 0.35 n 9 45 45 19 42
 25 45 45 28 28

III mean 9.4 16.6 21.7 2.71 - NP 1 54 18 6 COV 0.52 0.05
 0.16 0.004 - - 5.83 0.47 0.45 0.61 n 9 33 33 26 - - 34 34
 11 11

II mean 12.9 15.1 28.5 2.73 31.2 9.4 - - - - COV 0.58 0.05
 0.10 0.004 0.19 0.34 - - - - n 4 22 22 22 22 16 - - - -

I mean - - - - - - - - COV - - - - - - - - n - - -
 - - - - - - - -

Note: clay particle size <5 μ . distinct. Generally speaking, the clayey soil is relatively thin and the thickness of Layer IV is about 5-10 m. The sandy soil layers are relatively thick and the thickness of both Layer V and III is about 15-20 m. The sandy soil layers become thicker and the clayey soil layers become thinner as it comes closer to the Tamshui River. Descriptive statistics of physical properties of T2 soils is given in Table 4.

(3) Zone T3 - Zone T3 is located in the north of the Taipei Basin, close to the convergence of the Tamshui River and the Keelung River. The special characteristics of this Zone are relatively thick sandy soil and thinning out of the Layer VI. The relatively thick layers of clayey soil and the relatively thin or thinned-out sandy soil strata are the

characteristics of the drainage basin of the Keelung River. The thickness of Sungshan Formation in this

region is directly influenced by the elevation of the underlying rock formation. The drainage basin of

the Keelung River can be generally divided into two zones: Zone K1 and Zone K2.

(1) Zone K1 - Zone K1 includes the Sungshan Airport and the newly developed city center area, where the Taipei City Government and Taipei Financial Center (Taipei 101) are located. In this zone, Layer V is found to be in absence. Layer III in these areas is also very thin or non-existing. Consequently, The standard six layers of Sungshan Formation

is not applicable in this zone. The clayey soils from Layers II, IV, and VI become almost continuous and relatively thick. Descriptive statistics of physical properties of K1 soils is given in Table 5.

(2) Zone K2 - Zone K2 is in the northeastern part of the Taipei Basin, which is located in the downhill of the Tatun Volcanic Group. Similar to Zone K1, Zone K2 has a thick layer of clayey soil and lacks of the standard six-layer stratification. The average clay size particles content of the Layer VI and IV in Zone K2 is about 15% and 10%, respectively, which is higher than those in Zone K1. The average liquid limit and plasticity index of the Layer VI and IV are about 3 to 5% higher in Zone K2 as compared to those in Zone K1. The average liquid limit of Layer VI of this subzone reaches

Table 4. Descriptive statistics of physical properties of T2 soils (after MAA 1987). Grain size distribution (%) Statistical Thickness γ_d W n

Layer	items	(m)	(kN/m ³)	(%)	G	s	w	l	(%)	I	p	(%)	Gravel	Sand	Silt	Clay
VI	mean	4.5	14.5	31.2	2.72	35.8	12.9	-	10	58	32	COV	0.46	0.06	0.14	0.01
		0.17	0.37	-	0.74	0.18	0.36	n	251	545	559	419	433	415	389	388
		388	388	388	388											
V	mean	10.1	15.4	26.3	2.68	-	NP	1	75	19	4	COV	0.28	0.07	0.18	0.01
		-	-	5.79	0.19	0.62	0.78	n	259	1288	1316	996	-	-	950	950
		950	950	950	950											
IV	mean	9	14.3	32.1	2.72	34.3	12	-	8	61	31	COV	0.63	0.06	0.13	0.01
		0.15	0.37	-	1.06	0.15	0.38	n	258	1441	1463	1071	1085	1064	1068	1068
		1068	1068	1068	1068											
III	mean	10.6	16.1	23.9	2.69	-	NP	-	60	34	7	COV	0.55	0.06	0.18	0.01
		-	-	0.29	0.82	0.69	n	225	1116	1135	766	-	-	760	761	761
		760	761	761												
II	mean	8.4	15.5	27.2	2.72	30.3	9.2	0	9	67	25	COV	0.57	0.06	0.15	0.004
		0.16	0.36	-	0.81	0.12	0.36	n	181	951	957	657	683	608	651	651
		651	651	651												
I	mean	4.5	17.0	20.3	2.69	-	NP	1	62	29	7	COV	0.73	0.07	0.21	0.01
		-	-	5.10	0.28	0.51	0.70	n	112	273	281	183	-	-	184	184
		184	184													

Note: clay particle size <5 μ .

Table 5. Descriptive statistics of physical properties of K1 soils (after MAA 1987). Grain size distribution (%)
Statistical Thickness γ d W n

Layer items (m) (kN/m³) (%) G s w l (%) I p (%) Gravel
Sand Silt Clay

VI mean 4.3 14.3 32.1 2.71 34.1 11.1 0 11 61 29 COV 0.7
0.08 0.17 0.01 0.18 0.43 - 0.74 0.1 0.42 n 186 494 496 300
310 310 302 302 302 302

V mean 6.6 17.0 20.1 2.68 - NP 2 73 18 6 COV 0.4 0.12 0.35
0.01 - - 2.04 0.15 0.45 0.71 n 152 461 474 285 - 230 230
230 230 230

IV mean 20 13.7 35.1 2.72 35.2 12.9 0 4 61 35 COV 0.29 0.07
0.14 0.004 0.15 0.35 - 1.13 0.15 0.31 n 207 2557 2837 1474
1530 1488 1459 1457 1457 1457

III mean 3.6 16.1 23.5 2.69 - NP 0 54 36 11 COV 1.15 0.07
0.18 0.01 - - - 0.30 0.37 0.56 n 113 207 211 127 - - 128
128 128 128

II mean 9 15.2 28.3 2.71 33 12 0 12 54 35 COV 0.63 0.09
0.22 0.01 0.2 0.42 - 0.97 0.23 0.41 n 183 903 910 489 521
478 508 507 507 507

I mean 4.7 16.8 20.9 2.68 - NP 0 59 32 9 COV 0.69 0.05 0.19
0.01 - - - 0.25 0.41 0.56 n 91 233 245 128 - - 124 124 124
124

Note: clay particle size <5 μ .

67%, with average plasticity index of about 35%, and
natural water content around 60%. Descriptive

statistics of physical properties of K2 soils is given in
Table 6.

According to the depositional environment, the Hsintien
River Drainage Basin can be divided into two

zones: Zone H1 and Zone H2.

(1) Zone H1 - It is located on the alluvial valley plain
which is deposited by the Chingmei Creek, a

tributary of the Hsintien River. The subsoil is mainly
composed of clayey soil of about 20 to 30 m

Table 6. Descriptive statistics of physical properties of K2 soils (after MAA 1987). Grain size distribution (%) Statistical Thickness γ d W n

Layer	items	(m)	(kN/m ³)	(%)	G	s	w	l	(%)	I	p	(%)	Gravel	Sand	Silt	Clay																	
VI	mean	4.8	12.7	40.6	2.73	38.9	15.6	0.3	53	44	COV	0.57	0.07	0.14	0.1	0.16	0.32	-	1.21	0.19	0.26	n	88	1508	1539	784	895	886	871	869	869		
V	mean	4.9	14.1	33.1	2.67	-	NP	1	76	19	5	COV	0.74	0.10	0.23	0.01	-	-	3.91	0.23	0.71	0.96	n	53	188	198	119	-	-	125	126	125	125
IV	mean	26.9	12.8	40.5	2.73	38.8	15.5	0.3	53	45	COV	0.22	0.08	0.14	0.01	1.01	0.32	-	1.23	0.19	0.39	n	88	1654	1669	858	980	970	956	1495	954	955	
III	mean	4.2	15.6	26.4	2.69	-	NP	3	60	26	10	COV	0.61	0.12	0.30	0.01	-	-	1.98	0.26	0.46	0.72	n	70	127	132	68	-	-	85	85	85	85
II	mean	7.4	14.3	31.2	2.70	33.7	12	0.9	58	33	COV	0.95	0.08	0.2	0.01	0.19	0.41	-	1.05	0.18	0.33	n	28	95	102	48	53	53	47	47	47	47	
I	mean	3.9	16.6	21.2	2.70	-	NP	4	68	18	9	COV	0.67	0.08	0.26	0.01	-	-	2.11	0.17	0.43	0.96	n	15	20	22	10	-	-	10	10	10	10

Note: clay particle size <5 μ . in thickness directly overlying rock formation. In areas along the Chingmei Creek, the subsoils are more variable and composed of interlayers of silty sand and silty clay. Within the silty sand stratum, gravel particles are sometimes found.

(2) Zone H2 - The subsoils are mainly composed of sandy soil and gravel. The thickness of the clayey soil tends to increase from the Hsintien River eastwards. Descriptive statistics of physical properties of the H2 soils is given in Table 7.

3.2 Stratigraphy

Based on data collected during the construction of the priority network of the TRTS (Figure 2), the

stratigraphy of the Taipei Basin was studied (Chin et al. 1994). It is noted that most of the deeper

excavations for the TRTS projects is located in the T2 and K1 Zones. The stratigraphy in the T2 zone

is relatively uniform, consisting of six alternating layers of clays and sands. The uppermost Layer VI

together with Layers IV and II are cohesive soils, while Layers V and III are silty sands. The lowermost

Layer I is variable and contains both clayey and sandy materials. In the K1 Zone, sandy layers are thin

or absent and the profile is dominated by cohesive soils. A typical east-west section (Figure 10) along the TRTS Panchiao-Nankang Line (from Panchiao to

Nankang) illustrates the general change in stratigraphy from alternating clayey/sandy layers in the T2

Zone to the thick clay deposits in the K1 Zone to the east. Figure 11 shows a typical north-south section

along the TRTS Tamsui-Hsintien Line (from Peitou to Hsintien). The northern section of the line is in

K2 Zone, where all soil deposits are cohesive soils. Typical T2 stratigraphy occurs in the center section

of this line. A promontory of sandstone and tuff abruptly delineates the T2 Zone from the gravelly H2

Zone that occupies the southern section of the line. For the TRTS projects, Layer IV of the Sungshan deposits frequently occurs at about the invert of

open excavations and is the medium through which tunnels will pass in the T2 zones. In area nearby the

Taipei Main Station, this material is silty with low plasticity. To the east, it becomes more clayey and

construction performance in this area was dominated by deep deposits of soft clay. The strength of these

Layer IV materials has hence become focus of studies.

4 STATE AND YIELD BEHAVIOR OF TAIPEI CLAY

To investigate state and engineering properties of Taipei Clay, a Designated Task was carried out as part

of the GESG project of TRTS. Results obtained from this Designated Task provided principle materials

Table 7. Descriptive statistics of physical properties of H2 soils (after MAA 1987). Grain size distribution (%) Statistical Thickness γ d W n

Layer	items	(m)	(kN/m ³)	(%)	G	s	w	l	(%)	I	p	(%)	Gravel	Sand	Silt	Clay																	
VI	mean	5.6	13.9	33.0	2.71	35	13.8	0.8	57	35	COV	1.12	0.06	0.18	0.01	0.16	0.34	-	1.08	0.14	0.27	n	5	32	33	20	22	23	22	22	22	22	
V	mean	14.6	19.1	13.3	2.69	-	NP	22	52	20	5	COV	0.21	0.13	0.54	0.01	-	-	1	0.27	0.64	0.62	n	9	55	65	41	-	-	42	42	42	42
IV	mean	6.7	14.6	31.0	2.71	31.2	10.1	0	10	65	25	COV	0.86	0.07	0.16	0.01	0.16	0.31	-	0.79	0.12	0.40	n	10	61	62	37	26	42	41	41	41	41
III	mean	6.3	16.9	20.9	2.70	-	NP	2	68	25	7	COV	0.5	0.08	0.16	0.01	-	-	4.12	0.26	0.57	0.82	n	7	29	30	14	-	-	17	15	15	15
II	mean	-	14.4	30.8	2.71	33.2	9.5	0	16	66	18	COV	-	0.02	0.11	0.004	0.15	0.07	-	0.66	0.21	0.55	n	4	8	9	6	7	6	7	7	7	7
I	mean	-	17.2	21.3	2.70	-	-	0	80	18	2	COV	-	-	-	-	-	-	-	-	-	-	n	1	2	2	1	-	-	1	1	1	1

Note: clay particle size <5 μ . 0 2 3 km 040 010 020 030 010 20 070 060 050 040 010 020 030 0 10 20 Horizontal scale Tamshui River Taipei Main Station Silty clay Silty sand Gravel Sandstone Reduced level (m) West Panchiao East Nankang T2 Zone K1 Zone 070 060 050 1

Figure 10. Stratigraphy along the E-W Nankang Line (adapted from Chin et al. 1994).

to delineate engineering characteristics of the clay in this paper. In this section, the state properties of

Taipei Clay is presented primarily based on these results. To investigate the cohesive Sungshan deposits,

detailed and careful borings were put down at locations shown in Figure 2. Borehole R-1 is located

in the K1 Zone and Borehole R-2 is located in the T2 Zone, while Borehole R-3 is selected to be

in transition area of K1 and T2 Zones. These test sites were chosen to reflect the varying nature of

stratigraphy across the Basin, as suggested by Figure 10. At these sites, stationary piston sampling was

carried out continuously to retrieve undisturbed samples for laboratory testing. During drilling, the mud

in the borehole was controlled to have a unit weight of between 10.5 kN/m^3 and 11.0 kN/m^3 to reduce

disturbance resulted from swelling. At the K1 site, a section of samples were sent to Golder Calgary for 0 321 km 50 040 010 020 030 0 10 20 30 050 040 010 020 030 0 10 20 30 Reduced level (m) Horizontal scale Taipei Main Station Silty clay Silty sand Gravel Sandstone Tuff Shale South Hsintien North Peitou T2 Zone K2 Zone H2 Zone

Figure 11. Stratigraphy along the N-S Hsintien Line (adapted from Chin et al. 1994).

studying the state and yield surface of Taipei Clay. A portion of undisturbed tube samples from depth

18 m to 22 m were sent to MIT for a special laboratory testing program to study stress-strain-strength

behavior of the clay (Germaine 1992). All the other tests were performed at MAA laboratory in Taipei.

Results of this Designated Task have been summarized in Chin et al. (1994), Hu et al. (1996), and Chin &

Liu (1997).

4.1 State of Taipei Clay

To understand the fundamental strength properties, the concept of state has been used to evaluate behavior

of the cohesive Sungshan deposits (Chin et al. 1994). The strength behavior of a soil is controlled in

a first order sense by its void ratio and the effective stresses to which it is subjected. The current state

of the soil is represented as a point on a void ratio - effective stress plot. Current state is quantified by

its distance from a reference line. For sands, the steady state line (SSL) is used as a reference, and the

current state is quantified as the void ratio difference between the current state and the SSL at the same

stress level (i.e. state parameter ψ). Details of the basic state parameter concept is referred to Been &

Jefferies (1985). Chen et al. (1993) presents the results of the study on state parameters and related

engineering properties of Taipei silty sand. For clays, the virgin consolidation line (VCL) is used as a

reference, and current state is quantified as the ratio of the current stress to the maximum past pressure

(OCR). The use of OCR to represent the state of clays is the basis of the SHANSEP (Stress History and

Normalized Soil Engineering Properties) concept (Ladd & Foott 1974). Since the state of clays is defined in terms of OCR, accurate measurements of the following effective

stresses are required:

(1) Vertical in-situ stress (σ'_{vo}) This is more difficult in Taipei than is often the case because of the current complex groundwater conditions caused by past pumping from the Chingmei gravels and incomplete recovery of water pressures since pumping stopped. Total stresses are readily calculable but piezometers are required to define the in-situ water pressure. An estimate of σ'_{vo} can also be obtained from conventional oedometer tests using the "work/unit volume" approach proposed by Becker et al. (1987).

(2) Vertical yield stress (σ'_{vy}) These values are estimated from the results of oedometer tests. However, for the cohesive Sungshan deposits, the e log stress curves from oedometer tests are rounded with indistinct yield points. Hence, interpretation of these data is best carried out using the "work/unit volume" approach.

	10	20	30	40
Water content (%)	20	40	60	80
W _L (%)	10	20	30	40
I _P (%)	10	20	30	40
Gradation (%)	19	20	25	30
γ_t (kN/m ³)	2.7	2.8	2.9	3.0
Specific gravity	Legend : Sand	Silt (<0.075mm)	Clay (<0.005mm)	40 35

30	25	20	15	10	5	0	Depth (m)	Tube No.	USCS	T -01	T -02	T -03	T -04	T -05	T -09	T -06	T -07	T -08	T -10	T -11	T -12	T -13	T -14	T -18	T -15	T -16	T -17	T -21	T -22	T -23	T -24	T -25	T -26	T -27	T -28	T -29	T -30	T -31	T -32	T -33	T -34	T -35	T -36	T -37	T -38	T -39	T -40	T -41	T -42	T -43	T -44	T -45	T -46	T -47	T -48	T -49	T -50	T -51	T -52	T -53	T -54	T -55	T -56	T -57	T -58	T -19	T -20	CL	SM	CL
----	----	----	----	----	---	---	-----------	----------	------	-------	-------	-------	-------	-------	-------	-------	-------	-------	-------	-------	-------	-------	-------	-------	-------	-------	-------	-------	-------	-------	-------	-------	-------	-------	-------	-------	-------	-------	-------	-------	-------	-------	-------	-------	-------	-------	-------	-------	-------	-------	-------	-------	-------	-------	-------	-------	-------	-------	-------	-------	-------	-------	-------	-------	-------	-------	-------	----	----	----

Figure 12. Index properties of K1 soils (after Chin et al. 1994).

(3) Horizontal in-situ and yield stresses (σ'_{ho} and σ'_{hy})

These values are estimated using the “work/unit volume” interpretation of the results of oedometer

tests carried out on vertically trimmed samples.

4.1.1 K1 zone

Figure 12 shows the soil profile at the location of Borehole R-1, which reflects the typical strata patterns

as expected for the K1 Zone. Now the location of the Borehole R-1 is the Taipei City Hall Station of

TRTS Nankang Line. At this site, the soil is predominated by silty clay except for a 4-m thick silty

sand at about 30 m within the first 45 m of deposits below the ground surface. The nature of the clayey

soil changes slightly but consistently with depth, where the clay content increases down to about 15 m

and then decreases again below this depth. This variation is reflected in the index properties shown in

Figure 12.

Piezometric pressures were determined from inspection of piezometers installed in the boring and in

other borings drilled previously at nearby locations. While the latter show some variation with time, a

reasonably definitive linear trend with depth can be drawn from the most likely values. No information is

available above a depth of 15 m, and it is assumed that the current piezometric profile continues upward

until it intersects the hydrostatic line associated with a groundwater level at a depth of 1 m. The current

piezometric pressure profile is shown in Figure 13 by the solid line, and this would represent the upper

bound of piezometric pressure.

Based on the measured piezometric profile and calculated total stresses, the current in-situ vertical

effective stress (σ'_{vo}) profile can be accurately determined, as shown in Figure 13. The σ'_{vo} values estimated

from vertically loaded oedometer tests are reasonably consistent with the calculated σ'_{vo} profile. The

profiles of vertical and horizontal effective yield stress (σ'_{vy} and σ'_{hy}) and other state parameters (K_o

and OCR) with depth also are shown in Figure 13. Above 15 m depth, the σ'_{vy} values are approximately 100 200 300 400 500 Piezometric pressure (kPa) 100 200 300 400 500 600 700 800 Vertical effective stress (kPa) 100 200 300 400 500 600 Horizontal effective stress (kPa) 1 3 4 5 7 OCR $v = \sigma'_{vy} / \sigma'_{vo}$ 0.5 1.0 1.5 2.0 2.5 K_o Depth (m) 45 40 35 30 25 20 15 10 5 0 Original Current Distribution corresponding to σ'_{vy} Current σ'_{vo} Max. past pressure σ'_{vy} CRS test results Probable lowest piezo. CRS test results σ'_{vy} σ'_{vo} Work/Unit vol. interpretation of oedometer tests σ'_{hy} σ'_{ho} Legend : Piezometers in BHR-1 Piezometers near BHR-1 2 6 8

Figure 13. State properties of K1 soils (after Chin et al. 1994).

constant but are relatively high given the depth. The σ'_{hy} values are as high as σ'_{vy} . OCR v values also are

high at shallow depth, but it decreases to about 1.5-2 at about 15 m deep. Similar trend also is seen for

the K_o profile, which has high values above 15 m depth. The state data below 15 m can be readily understood in terms of known stress history of the deposit.

Therefore, the σ'_{vy} values form a consistent trend and

lie above the current σ'_{vo} profile, and the magnitude of overconsolidation is consistent with a past piezometric profile that reflects a 35-40 m water level lowering in the underlying Chingmei gravels. As a result, OCR decreases slightly from about 1.7-1.4 with depth. Given the stress history below 15 m depth, the K_o values that range from 0.45 to 0.66 appear reasonable. Nevertheless, unrealistically low values were measured at depths and have been ignored.

4.1.2 T2 Zone

The stratigraphy and index property data of Borehole R-2 are shown in Figure 14. Borehole R-2 soil profile is typical of the T2 Zone, where all of Layers I to IV can be easily recognized. The piezometric pressure profile and other state properties are given in Figure 15. The piezometric conditions in the depth range for Layers IV to VI are well defined based on the two piezometers installed in Layer IV and the piezometric pressures measured in Layer V at the end of a companion CPT dissipation test. It is found that the measured piezometric profile is consistent with other data in the general T2 Zone. Based on these data, the state characteristics of the two silty clay layers can be determined. The 3 m thick Layer VI silty clay is moderately overconsolidated based on the oedometer test results. However, the degree of overconsolidation is not as high as in the upper portion of the K1 Zone. Layer IV consists of a moderately plastic clayey silt to silty clay material that is typical throughout the T2 zone. Two sets of oedometer tests indicated that this layer is slightly overconsolidated with an OCR of 1.6. The magnitude of overconsolidation above the current σ'_{vo} profile is consistent with the fact that

the piezometric pressure at the top of the underlying sandy layer was zero during the period of maximum

groundwater lowering when pumping took place from the lower Chingmei gravels. In addition, the σ'_{ho}

values from the horizontally loaded oedometer tests provide reasonable K_o values of about 0.5. 10 20 30 40 Water content (%) 20 w L (%) 10 20 I P (%) 10 20 30 40 Gradation (%) 2.7 2.8 2.9 3.0 Legend : Sand Silt (<0.075mm) Clay (<0.005mm)

40

35

30

25

20

15

10

5

0

Depth

(m) Tube No. USCS T - 01 T - 02 T - 03 T - 04 T - 05 T - 09 T - 06 T - 07 T - 08 T - 10 T - 11 T - 12 T - 13 T - 14 T - 18 T - 15 T - 16 T - 17 T - 21 T - 19 T - 20 T - 22 T - 25 T - 23 T - 24 T - 27 T - 26 T - 30 T - 28 T - 29 T - 31 T - 32 T - 34 T - 33 T - 37 T - 35 T - 36 T - 38 S - 39 S - 40 S - 41 CL (VI) SM/ML (V) CL (IV) SM/ML (III) CL (II) SM (I) Silty Silty Silty 40 γ_t (kN/m³) 1920 Specific gravity

Figure 14. Index properties of T2 soils (after Chin et al. 1994).

4.1.3 K1/T2 transition zone

The stratigraphy and index property data of Borehole R-3 are shown in Figure 16. Borehole R-3 is located

at the transition of T2 and K1 zones. The typical six-layer

Sungshan deposit profile is still present but

with thick cohesive layers compared to that in the T2 zone. In addition, Layers I to III at this location are

poorly defined below 29 m in depth, and this zone is best described as silt - clayey silt with silty sand

interbeds.

The piezometric profile of this test site is shown in Figure 17, and it is reasonably well defined by

piezometers installed in this borehole and others previously installed in nearby locations. The results

are very similar to those at the R-1 site. For the state characteristics, Layer VI is over-consolidated with

high K_o values as is the case for the K1 location. Furthermore, for Layer IV clay, six pairs of vertically

and horizontally loaded oedometer tests were conducted, and the results provide a good definition of the

state of soils within this layer. As can be seen in Figure 17, the σ'_{vy} profile is reasonably consistent with

the maximum vertical effective stress calculated based on the maximum past groundwater lowering due

to pumping from the Chingmei gravels. The resulting OCR is approximately consistent at about 1.4-1.6,

and K_o values are consistently equal to about 0.6.

4.2 Yield behavior

Most cohesive soils exhibit both "small strain" and "large strain" behavior. Small strain behavior is

characterized by undrained strength in compression and to a lesser extent, in extension. A common

example of small strain behavior in a drained test is the yield stress evident in an oedometer test. Large Legend :
Piezometers in BH R-2 CPT measurements 0.5 1.0 1.5 2.0 2.5
 K_o 100 200 300 400 500 Piezometric pressure (kPa) 100
200 300 400 500 600 700 Vertical effective stress (kPa) 100
200 300 400 500 600 Horizontal effective stress (kPa) 1 2 3

4 OCR $v = \sigma'_{vy} / \sigma'_{vo}$ Depth (m)

45

40

35

30

25

20

15

10

5

0 Original Current Layer VI Layer IV Original σ'_{vo}
 Current σ'_{vy} Amount of overconsolidation consistent with
 zero piezometric pressure at top of layer III during past
 maximum groundwater lowering σ'_{vy} Work/Unit vol.
 interpretation of oedometer tests σ'_{vo} σ'_{ho} σ'_{hy}

Figure 15. State properties of T2 soils (after Chin et al. 1994).

strain behavior occurs in undrained strength tests when the material state reaches the ϕ' line. In an

oedometer test, large strain behaviour occurs at stresses in excess of the yield stress (i.e. the normally

consolidated stress range). The total stress paths imposed in triaxial strength tests and oedometer tests are limited by the test

conditions. However, under field loading conditions, the soil can be loaded along a wide variety of stress

paths depending on the nature of loading. While the response of the soil to varying stress paths will vary

in detail, there is a distinction between small and large strain behavior regardless of stress path. Small strain behavior under various stress paths can be investigated in the laboratory by carrying out

drained triaxial tests along various stress paths. For each stress path in q - p' stress space, a yield point

can be identified which indicates the boundary between small strain and large strain behavior. The locus

of these yield points in q - p' stress space is termed the yield envelope. Small strain behavior occurs at

stress states within the yield envelope, while large strain behavior occurs outside the yield envelope. The undrained compression strength will coincide with the upper surface of the yield envelope while

the extension strength will coincide with the lower surface of the yield envelope. The apex of the yield

envelope is typically defined by the preconsolidation pressure and the axis of the envelope will be approx

imately symmetrical around the K_o line. Except for bonded or highly structured soils, the large strain

lines in compression and extension will define the upper and lower limits of the yield envelope at lower

stress levels; by definition stress states beyond the ϕ' lines in compression and extension cannot exist. The yield envelope concept represents a simple qualitative stress-strain-strength model for lightly over

consolidated soils. It is useful for understanding the behaviour of soils under field loading (Folkes &

Crooks 1985, Crooks et al. 1984). It is also useful for evaluating the consistency of laboratory and field

strength data. 10 20 30 40 Water content (%) 20 wL (%) 10 20 IP(%) 10 20 30 40 Gradation (%) 2.719 20 2.8 2.9 3.0 Specific gravity Legend : Sand Silt (<0.075mm) Clay (<0.005mm)

40

35

30

25

20

15

10

5

0

Depth

(m) Tube No. USCS T - 01 T - 02 T - 03 T - 04 T - 05 T - 09
T - 06 T - 07 T - 08 T - 10 T - 11 T - 12 T - 13 T - 14 T -
18 T - 15 T - 16 T - 17 T - 21 T - 19 T - 20 T - 22 T - 25
T - 23 T - 24 T - 27 T - 26 T - 30 T - 28 T - 29 T - 31 T -
32 T - 34 T - 33 T - 37 T - 35 T - 36 T - 38 T - 39 T - 40
T - 41 T - 53 T - 52 T - 56 T - 54 T - 55 T - 57 T - 58 T -
46 T - 45 T - 49 T - 47 T - 48 T - 50 T - 51 T - 44 T - 43
T - 42 CL (II) CL (VI) SM/ML (V) CL (IV) SM/ML (III) 40 γ t
(kN/m³)

Figure 16. Index properties of K1/T2 soil (after Chin et al. 1994).

4.2.1 Yield envelopes for cohesive Sungshan deposits

The yield envelopes for samples of the cohesive Layer IV Sungshan deposits from each of the three

locations were determined in drained triaxial tests. The test samples were reconsolidated to their in-situ

stresses prior to testing. Because the samples were obtained from different depths, the yield data were

normalized with respect to their respective in-situ vertical yield stresses. The yield data from these tests

are shown on Figure 18 and describe a consistent generalized yield envelope for the Layer IV deposits.

Data from other tests also fit well with the generalized yield envelope:

- The results indicate a consistent yield behavior regardless of sample origin (i.e. the normalized yield envelope for each material type is essentially the same).

- The compression and extension strengths predicted based on normalized (SHANSEP) strength behavior using a typical OCR value of 1.6 correlate closely to the upper and lower portions of the generalized yield envelope.

- The apex of the generalized yield envelope is reasonably well defined by the yield stress measured in

oedometer tests and is consistent with a K_o value of 0.55.

- The large strain effective stress lines clearly provide upper and lower limits for the generalized yield

envelope.

5 ENGINEERING PROPERTIES OF TAIPEI CLAY

To understand the fundamental behavior of Taipei Clay, high quality continuous piston samples were

taken from K1 zone for advanced laboratory tests in the Designated Task of GESC, TRTS. In that study, 100 200 300 400 Piezometric pressure (kPa) 100 200 300 400 500 600 700 800 Vertical effective stress (kPa) 100 200 300 400 500 600 Horizontal effective stress (kPa) 1 432 5 6 7 8 OCR $v = \sigma_v / \sigma_v' = 0.5 \ 1.0 \ 1.5 \ 2.0 \ 2.5$ K_o Depth

(m)

40

35

30

25

20

15

10

5

0 Probable max. past drawdown Current σ_v' v0 Max. past σ_v' v0 for drawdown Current Original Silty Silty Original σ_v'

vo Silty Silty Based on max. past σ' vo for drawdown Ave.
measured σ' vy σ' hy σ' vo σ' ho Work/Unit vol.
interpretation of oedometer tests Legend : Piezometers
in BH R-3 Piezometers near BH R-3 (distance<100cm)
Piezometers near BH R-3 (distance>100cm)

Figure 17. State properties of K1/T2 soil (after Chin et al. 1994).

constant-rate-of-strain (CRS) consolidation tests were conducted to study the volumetric behavior under

normally and over-consolidated states. K o -consolidated undrained triaxial compression, direct simple

shear, and triaxial compression tests were conducted using SHANSEP procedures at various OCRs.

These test results provide insightful information on undrained behavior of Taipei Clay. Along with the

other studies, engineering properties of Taipei Clay are discussed in this section.

5.1 Sample preparation and test procedures

This section presents a summary of sample preparation procedures at the Geotechnical Laboratory at

MIT (Germaine 1992). The testing program conducted at MIT also are summarized in this section.

5.1.1 Sample preparation

For the special laboratory testing program, four 10 cm (3 inch) diameter by 76.2 cm (30 inch) long

Shelby tube samples taken from Borehole R-1 were sent to the Geotechnical Laboratory at MIT for

compressibility and strength tests. The specimens were first examined using radiography and the best

quality; most uniform sections were selected for preparing test specimens. Figure 19 shows an example

of the X-ray examination film (Germaine 1992). Specimens were prepared for all engineering tests

following very careful procedures, and those of best

quality were selected for CRS tests. The tubes

were cut above and below the selected specimens to reduce disturbance due to extrusion. The remaining

segments of the tube were sealed for later use. The specimen was then extruded from the tube using the

following procedures:

1. A fine wire was penetrated through the soil along the inside of the tube.
2. The wire was used to cut the soil free from the inside perimeter of the tube
3. The soil was gently pushed by hand free of the tube. The resulting block of soil was trimmed in different manners depending on the specific type of tests. In

this study, a total of six triaxial, one direct simple shear, and two CRS consolidation tests were performed.

Figure 18. Yield envelope for Layer IV soils (after Chin et al. 1994).

Figure 19. X-ray examination result of R-1 sample (after Germaine 1992).

5.1.2 Testing program

CRS consolidation tests were performed using MIT device that was developed by Wissa (1971). However,

modification to the original apparatus was made by removing the inner diaphragm seal to reduce the

potential for friction errors. The specimen is 5 cm in diameter and 2 cm in height. After assembling

the test equipment, the specimen was saturated by backpressure at constant volume to about four atmo

spheres (390 kN/m²). Then consolidation was performed by adding vertical pressure at a fixed strain rate

Table 8. Summary of consolidation tests of Taipei Clay (after Chin & Liu 1997).

Depth Test w n u b e σ'_{vo} c v σ'_{vm}

(m) no. (%) e_o (kPa) (sec⁻¹) (kPa) CR RR (cm²/sec)
(kPa)

19.90~19.96 CRS66 38 1.07 393 1.4×10^{-6} 193 0.175 - 3.7×10^{-3} 280 $\sim 6.3 \times 10^{-3}$

19.35~19.38 CRS68 40 1.10 392 1.44×10^{-6} 188 0.180 0.02
 1.5×10^{-3} 300 $\sim 2.3 \times 10^{-3}$

18.50~18.54 OED44 42 1.17 - - 181 0.183 - 1.3×10^{-3} 240

of about 0.5%/hr. Under normal condition, the ratio of the measured base pore water pressure over the

vertical stress shall not exceed 0.05. Triaxial tests were performed using a computer controlled system and procedures developed at MIT.

The triaxial cell features a low friction rolling diaphragm seal, a fixed top cap geometry and double

drainage. The test specimen is about 8 cm tall and 10 cm² in cross section. All measurements were made

externally using electronic transducers. The specimen is enclosed in two prophylactic membranes. Radial

drainage is provided by vertical filter drains for compression tests and spiral drains for extension tests.

Silicone oil was used for the cell fluid to prevent membrane leakage. Data were corrected for membrane

resistance and filter drain resistance in compression tests. For the six triaxial tests, SHANSEP procedures

were followed. Undrained tests with OCRs = 1, 2, and 4 were performed. The specimen was consolidated

to the desired maximum stress state (usually 2 to 4 times the estimated maximum past pressure) using a

constant rate of deformation at about 0.07 %/hr and controlled by computer to maintain the K_o condition

or a specific stress path. The maximum stress is maintained for 24 hours. When appropriate consolidation

was done, the specimen was rebounded to the desire OCR and

controlled by computer to maintain the

final stress for at least 8 hours. Before shearing, drainage lines were closed and the change in pore water

pressure was monitored for 30 minutes to confirm that the specimen is sealed and the rate of secondary

compression is small. Finally, the specimen was sheared at a constant rate of axial deformation ranging

from 0.2%/hr to 0.5%/hr to failure. This rate was estimated to be slow enough such that dilation will not

occur during shearing. The basic Geonor direct simple shear device was used at MIT, and the device is fully automated to

allow precise control during both consolidation and shearing phase of the test. The specimen is prepared

to have a cross section area of 35 cm² and 2 cm height. SHANSEP type of procedures were followed for

the testing. The specimen was consolidated at a constant rate of axial deformation of about 0.5 %/hr to a

predetermined maximum vertical consolidation stress, and the maximum stress is held constant for about

24 hours or one cycle of secondary compression. Undrained shear was performed at a constant rate of

horizontal deformation of about 5%/hr. Undrained conditions are maintained by computer control of the

vertical stress to ensure constant specimen, and the computer automatically adjusts for the compliance

of the apparatus.

5.2 Compressibility and permeability

Two K1 Layer IV samples were tested under CRS consolidation tests to study the compressibility and

permeability characteristics of Taipei Clay (Germaine 1992). In addition, another sample was tested

using conventional oedometer apparatus (Chin & Liu 1997).

The summary of these consolidation tests

is given in Table 8, and Figure 20 shows the stress-strain relationship of these tests.

5.2.1 Compressibility

Figure 19 shows very consistent CRS consolidation test results for the clay, and it also shows that

the yield point (or the maximum past consolidation stress, σ'_{vm}) can be clearly depicted. Both CRS test

results also shows a linear normal consolidation line, which suggests that soil particles were not deflected

and the clay is not highly structured. Therefore, the compressibility during normal consolidation can be

represented by a single compression index (C_c), and the values for these two CRS tests are 0.36 and 0.38.

Since compressibility of soil is a function of the void ratio, the compression ratio [$CR = C_c / (1 + e_0)$] is

a better index for comparison. The CR value for the Taipei obtained from the CRS test is about 0.18,

which is very comparable to that of the Boston Blue Clay ($CR = 0.17$, Kavvasdas 1982). The compressibility of clay under overconsolidation condition can be analyzed through results of

rebound and recompression loadings. Figure 20 demonstrate that hysteretic behavior during rebound Vertical Effective Stress, σ'_{vc} (kPa) A x i a l S t r a i n , ϵ_a (%)

Figure 20. Stress-strain relationship for Taipei Clay under CRS and oedometer tests (after Chin & Liu 1997).

and recompression exists for Taipei Clay. In traditional soil mechanics, the rebound and recompression

behavior is related to OCR. However, if the true hysteretic response is to be considered, it is not enough

using OCR to describe the rebound and recompression behavior at the same time. Instead, the concept

of stress ratio has to be introduced (Whittle et al. 1993).

The stress ratio (ξ) is defined as follows:

in which σ'_{rc} is the rebound or recompression stress that the soil experienced and σ'_o is the consolidation

stress at the beginning of the rebound or recompression process. Based on these definition, ξ is equal

to OCR during rebound, however it is only a dimensionless index during recompression. Nevertheless,

this stress ratio concept is important for establishing a comprehensive soil model.

The incremental vertical strain during both rebound and recompression stages are plotted versus the

stress ratio in Figure 21 (Whittle et al. 1993). It is noted that the incremental strain has been corrected

for the amount induced by the secondary compression in this plot. As can be seen from this plot, values

of swelling ratio (SR) and rebound ratio (RR) are not constant, but increase with increasing ξ . It is

also evident that SR and RR values for both rebound and recompression stages are very close, and this

suggests that the hysteretic loop is symmetrical. When the value of ξ is between 1 and 3.5, both SR and

RR values are approximately smaller than 0.02. In addition, the overconsolidation compressibility of the

Taipei Clay is very close to the Boston Blue Clay, as shown in Figure 21.

The results of CRS tests can be compared to that of the traditional odometer test. For the maximum

past consolidation stress (σ'_{vm}), results obtained from the CRS tests are approximately 280~ 300 kPa.

The σ'_{vm} value is about 240 kPa for the odometer test, which is about 14% to 20% smaller than that for

the CRS tests. Leroueil et al. (1983) showed that for soils taken from the same depth, values of σ'_{vm} are

higher for tests under higher strain rates, and he concluded that the σ'_{vm} value obtained using the CRS

test is approximately 13% ($\pm 9\%$) greater than that obtained by the odometer test. With reference to this

study, results for Taipei Clay are quite reasonable. For the compression index, C_c (OED) is about 0.4, while

C_c (CRS) ranges from 0.36 to 0.38. However, in terms of CR, all the results are very comparable, and the

value is about 0.18. Under overconsolidation condition, the compressibility suggested by the odometer

test is very close to that of the CRS test, as shown in Figure 20.

5.2.2 Permeability

The vertical coefficient of permeability (k_v) from the results of CRS tests can be computed based

on suggestions given by Wissa et al. (1971). The computed vertical coefficient of permeability from

these two CRS tests are plotted versus the void ratio in Figure 22. The estimated value of in-situ k_v is 1.02×10^{-6} cm/sec. Stress ratio, $\xi = 4.2$. Incremental strain, $d\varepsilon_a (\%)$. $SR = RR = 0.01, 0.02, 0.03, 0.04, 0.05$. Boston blue clay ($IP = 21\%$). Test type Rebound Recompression. CRS Symbol 370 760 200 220. Test type Rebound Recompression OED Symbol 400 100. σ'_o (kPa) σ'_o (kPa)

Figure 21. Compressibility of Taipei Clay under overconsolidation (after Whittle et al. 1993).

is about 0.6 to 1.2×10^{-7} cm/sec. These values are consistent with results from multi-stage constant

head triaxial permeability tests reported by Whittle et al. (1993), which showed that k_v values of Taipei

Clay at Borehole R-1 range from 1 to 2×10^{-7} cm/sec. As can be seen, there is an approximate linear

relationship between $\log-k$ and the void ratio. In general, the coefficient of permeability decreases with

decreasing void ratio. From these two tests, values of the

slope (C_k) of the e -log k lines are 0.55 and

0.58, respectively. Tavenas et al. (1983) studied the permeability of various clays and suggested that C_k

can be expressed by the equation given below:

in which e_o is the initial void ratio. The estimated values of C_k using Equation 3 for the two CRS tests

are 0.54 and 0.55, which are very close to the measured results.

5.3 Undrained shear behavior

5.3.1 Normalized behavior

Based on a series of CIUC tests, it is found that the undrained shear strength of Taipei Clay can be

described by the SHANSEP equations. Moreover, the normalized undrained shear strength (s_u/σ'_{vc}) of

Taipei Clay is a function of plasticity index (I_p), and the results are shown in Figure 23. Chin et al. (1989)

also summarized effects of anisotropy and shearing modes on the undrained shear strength for Taipei

Clay based on tests conducted at MAA, and the results are shown in Figure 24. It should be noted that

the predetermined K_o was used for these CAUC and CAUE tests.

5.3.2 K_o -triaxial compression and extension tests

Strength tests of the Designated Tasks carried out at MIT included six undrained triaxial shear tests.

In all of the triaxial tests, specimens were consolidated to various OCRs (OCR's = 1, 2, and 4). The

K_o -consolidated specimens then were sheared in compression and extension modes (CKoUC and

CKoUE). The basic information of these six triaxial shear tests is tabulated in Table 9. Permeability, k (cm/sec) 0.5
0.7 0.9 1.1 1.3 V o i d r a t i o , e 0.55 1 0.58 1 10^{-9}
10 -8 10 -7 10 -6 CRS66 $e_o = 0.7$ CRS68 $e_o = 1.0$

Figure 22. Relationship between coefficient of permeability and void ratio for Taipei Clay (after Chin & Liu 1997).
 102 4 6 8 OCR 0 0.5 1 1.5 2 2.5 3 3.5 s_u / σ'_c No. I P (%) 1 2 3 4 5 7 10 15 20
 25 1 2 3 4 5

Figure 23. Relationship between undrained shear strength and OCR for Taipei Clay tested under CIUC (after Chin

et al. 1989). 0.4 0.6 0.8 1 1.2 $(s_u / \sigma'_{vc}) / (s_u / \sigma'_{vc})$ CIUC 0 10 20 30 51 15 25 I P (%) CAUC CAUE

Figure 24. Effect of anisotropy and shearing modes on undrained shear strength of Taipei Clay (after Chin et al.

1989).

Table 9. Results of triaxial compression and extension tests of K1 clays (after Germaine 1992). Result of consolidation Rate of

Depth Test w n consolidation σ'_{vm} σ'_{vc}

(m) no. (%) ϵ_o (%/hr) (kPa) (K o) NC (kPa) OCR (K o) OC
 Note

19.77-19.95 TRI148 36 1.01 0.08 640 0.52 640 1 - CK 0 UC

20.78-20.90 TRI158 39 1.08 0.08 700 0.55 700 1 - CK 0 UE

18.85-18.95 TRI159 38 1.08 0.07 690 0.56 339 2.03 0.78 CK 0 UC

19.59-19.69 TRI160 37 1.05 0.08 700 0.52 348 2 0.69 CK 0 UE

20.64-20.74 TRI193 37 1.02 0.11 700 0.52 174 4.04 0.93 CK 0 UE

20.49-20.59 TRI194 40 1.12 0.09 670 0.56 171 3.92 1.07 CK 0 UC Result of shearing Rate of Maximum axial stress Maximum obliquity shear

Depth (m) Test no. (%/hr) ϵ_a (%) q/σ'_{vc} p'/σ'_{vc} ϕ' (deg) μ ϵ_a (%) q/σ'_{vc} p'/σ'_{vc} ϕ' (deg)

19.77-19.95 TRI148 0.4 0.4 0.29 0.711 24.1 1 10 0.252 0.493 30.7

20.78-20.90 TRI158 -0.49 -13.3 -0.192 0.351 33.2 1 -13

-0.192 0.35 33.3

18.85-18.95 TRI159 0.16 1.7 0.51 1.127 26.9 0.2 6.5 0.484
1.012 28.3

19.59-19.69 TRI160 -0.32 -13.7 -0.329 0.57 35.2 0.79 -12.6
-0.326 0.561 35.4

20.64-20.74 TRI193 -0.5 -12.4 -0.633 1.607 36.4 0.41 -11.9
-0.631 1.056 36.7

20.49-20.59 TRI194 0.2 3.55 0.832 1.738 28.6 0.08 3.55
0.832 1.738 28.6 The relationship between K_o and OCR has
been established using results obtained from these triaxial

tests (Chin & Liu 1997). Figure 25 shows this relationship
for the four overconsolidated samples. As

can be seen from this figure and Table 9, the value of K_o
at NC state ranges from 0.52-0.58, and it

increases with the increasing OCR. The results from these
four tests fall in a small range. To describe the

K_o - OCR relationship, an empirical correlation based on
effective stress friction angle was suggested

by Mayne & Kulhawy (1982). When $\phi' = 28^\circ$ is used in this
empirical correlation, it can be seen that the

estimated results agree closely with the test results both
in trend and in value. 1 1 3 4 5 6 7 8 OCR 0.4 0.6 0.8 1.0
1.2 K_o $K_o = 0.54$ OCR 0.47 TRI 159 TRI 194 TRI 160 TRI
193

Figure 25. K_o -OCR relationship for Taipei Clay (after
Chin & Liu 1997).

The results of this series of tests are presented in Figure
26 as the stress-strain curves and in Figure 27

as the stress paths. Under normally consolidation
conditions, the values of undrained shear strength ratio

(s_u / σ'_{vc}) are 0.29 and 0.19 in compression and
extension, respectively. The friction angle in compression,

$\phi_{TC} = 28^\circ$ is well defined from effective stress
measurements at axial strain levels, $\epsilon_a \approx 10\%$. The

equivalent friction angle in extension, ϕ'_{TE} , is more difficult to estimate due to specimen necking. The

selected friction angle, $\phi'_{TE} = 33^\circ$ was estimated at $\epsilon_a \approx 10\%$ (Whittle et al. 1993).

The normalized undrained secant modulus (E_u / σ'_{vc}) are plotted versus the axial strain in Figure 28.

The degradation of modulus with increasing axial strain is apparent. For CK o UC test results, it is found

that the values of E_u / σ'_{vc} increase with increasing OCR at the same level of ϵ_a . For CK o UE tests, the

OCR influence is not obvious, which suggests that the effect of previous overconsolidation is greater on

vertical undrained modulus than on the horizontal one. Kung (2003) studied behavior of Taipei Clay at

small strain. Deformation modulus of samples taken from K1 Zone was measured using bender elements

in triaxial tests. It was found that the results of E_u / σ'_{vc} obtained from the small strain tests are close to

those shown in Figure 28 over the same axial strain range. When comparing the maximum shear modulus

obtained in laboratory and at site, it was found that the value measured by the bender element is slightly

lower than those measured by in-situ down-hole and cross-hole tests (Kung 2003).

5.3.3 Direct simple shear tests

In addition, a single undrained direct simple shear test (CKoUDSS) was also performed on a normally

consolidated specimen of the clay. The stress-strain curve of this test is shown in Figure 29, and the

stress path of this test is given in Figure 30. At a shear strain of about 10.2%, the maximum shear stress

is reached. The corresponding undrained shear strength ratio (τ_h / σ'_{vc}) is about 0.23.

Another set of direct simple shear tests were conducted by Liu (1999). In this study, tests were

performed at various overconsolidation ratios. The stress-strain relationship for these DSS tests are shown

in Figure 31, while the corresponding stress paths are given in Figure 32. For normally consolidated

sample, the MIT test result is very close to that obtained in this study. As can be seen, the undrained shear

strength of the clay is a function of OCR. These test results are useful for establishing the SHANSEP

equation, as discussed later.

Axial strain, ϵ_a (%)	-0.8	-0.6	-0.4	-0.2	0.0	0.2	0.4	0.6	0.8	1.0	1.2
$(\sigma_v - \sigma_h) / (2 \sigma_v)$	0.28	0.24	0.20	0.16	0.12	0.08	0.04	0.00	-0.04	-0.08	-0.12

CK o UC OCR=1
OCR=2 OCR=4 CK o UE OCR=1
OCR=2 OCR=4

Figure 26. Stress-strain curves for six triaxial tests of Taipei Clay (after Chin & Liu 1997).

$(\sigma_a' + \sigma_r') / 2$ (kN/m ²)	0	28	33	100	600	500	400	300	200
$(\sigma_a' - \sigma_r') / 2$ (kN/m ²)	0	28	33	100	600	500	400	300	200

$\phi' = 28^\circ$ $\phi' = 33^\circ$ CK o UE OCR=1 OCR=2 OCR=4
100 600 500 400 300 200
300 -50 -200 -100 -150 250 200 150 100 50 0 CK o UC OCR=1
OCR=2 OCR=4

Figure 27. Stress paths for six triaxial tests of Taipei Clay (after Chin & Liu 1997).

Axial strain, ϵ_a (%)	0	0.0001	0.001	0.01	0.1	1	10	100
E_u / σ_v	0	0.0001	0.001	0.01	0.1	1	10	100

CK o UC OCR=1 OCR=2 OCR=4
CK o UE OCR=1 OCR=2 OCR=4

Figure 28. Normalized undrained modulus degradation for six triaxial tests of Taipei Clay (after Chin & Liu 1997).

Figure 29. Stress-strain curve of Taipei Clay under CKoUDSS tested at MIT (after Germaine 1992).

5.3.4 SHANSEP strengths

For normalized strength behavior of Taipei Clay, undrained shear test results from samples obtained

at various locations were studied. At K1 site, samples were taken at sufficient depth to be below the

highly over-consolidated upper zone. Both compression and extension tests were performed (CK o UC

and CK o UE tests), and results were given in the previous section. Samples of Layer IV soils from various

depths and locations also were tested, and they were normally consolidated under K o conditions in triaxial

cells (CK o UC tests).

Figure 30. Stress path for Taipei Clay under CKoUDSS tested at MIT (after Germaine 1992). 0 4 8 12 16 20 Shear strain, γ (%) 0.0 0.2 0.4 0.6 0.8 1.0 1.2 1.4 1.6 $\tau_h / \sigma_v c'$ MIT test results OCR Sym 1 2 4 8

Figure 31. DSS stress-strain curve for Taipei Clay (adapted from Liu 1999). Results of compression tests were compiled in Figure 33, where the undrained shear strength ratio is

plotted versus OCR. The majority of these strength test data lie in a relatively narrow band regardless

of location. The relationship between the compression strength and OCR v can be established, and the

mean relationship can be expressed by the equation given below:

It is noted that some normally consolidated samples exhibit high normalized strengths. This phenomenon

can be attributed to dilation. For this type of effect, the SHANSEP approach does not apply, and these 0 0.4 0.8 1.2 0 0.1 0.2 0.3 0.4 $\tau_h' / \sigma_v m'$ MIT test results OCR Sym 1 2 4 8 σ_v' / σ_{vm}'

Figure 32. DSS stress paths for Taipei Clay (adapted from Liu 1999). 1 102 4 6 8 OCR v 0.1 1.0 0.2 0.4 0.6 0.8 2.0 $s_u / \sigma_v' c' s_u / \sigma_{vc}' = 0.32(OCR v) 0.82$ Symbol Borehole Tube no. T24/25 T17/18 T44 T21/22 T21/22 R1 R1 R1 R2 R3 R3 T26/27 Solid symbols : dilatant behavior during shearing

Figure 33. Undrained compression strength ratio versus OCR from CK o UC tests (after Chin et al. 1994).

data were excluded for interpretation of relevant normalized soil behavior. It should be noted that tests

run at MAA Laboratory is about 8.9 %/hr, which is about 17 to 55 times of that used at MIT. The pore

water pressure response during undrained shear in compression is shown in Figure 34, and the behavior

is typical for a wide range of clays except for those samples exhibit dilation.

For undrained extension, only two sets of tests were performed following the SHANSEP procedure,

and the results are shown in Figure 35. Nevertheless, consistent relationships between undrained shear

ratios and OCR still can be established. The correlations are shown in the figure and are given below for

convenience: 1 102 4 6 8 OCR v -0.5 0.0 0.5 1.0 1.5 2.0 A f
Symbol Borehole Tube no. R1 T17/18 R1 T44 R2 T21/22 R3
T21/22 R3 T26/27 Solid symbols : dilatant behavior during
shearing

Figure 34. Porewater pressure response during shearing from CKoUC tests (after Chin et al. 1994). 1 102 4 6 8 OCR v 0.1
1.0 0.2 0.4 0.6 0.8 2.0 s u / σ'_{vc} ' s u / σ'_{vc} '=0.19(OCR
v) 0.82 Symbol Borehole Tube no. R1
T17/18 R1 T44

Figure 35. Undrained compression strength ratio versus

OCR from CKoUE tests (after Chin et al. 1994). 1 102 4 6 8
OCR v 0.1 1.0 0.2 0.4 0.6 0.8 2.0 s u / σ'_{vc} Taipei clay
Borehole R-1 DSS s u / σ'_{vc} =0.23(OCR v)0.75 Figure 36.
Undrained compression strength ratio versus OCR from DSS
tests (after Chin et al. 1994).

For DSS tests, results shown in a previous section were studied. Figure 36 shows the plot of undrained

shear strength ratio versus OCR. The normalized behavior can be established, and the correlation can be

described as follow: 0 200 400 600 800 ($\sigma'_1 + \sigma'_3$)/2
(kPa) -200 0 200 400 ($\sigma'_1 - \sigma'_3$) / 2 (k P a)
Compression Extension Lab test result Borehole R-1
Borehole R-2 Borehole R-3 Based on effective stress states
at large strain in CK o U tests

Figure 37. Effective stress strength envelopes of Taipei Clay (after Chin et al. 1994).

5.3.5 Effective stress friction angle

The effective stress states at the end of consolidated undrained tests also provide information on the

effective stress friction angle of soils. Figure 37 compiles results of all K o consolidated undrained

triaxial tests both in compression and in extension. As can be seen, little scatter exists and the average

effective stress friction angle for all cohesive Sungshan deposits is about 32 degrees both in compression

and in extension.

5.3.6 Comparisons of results from different tests

To investigate the effect of different reconsolidation procedures on the undrained shear strength, tests

were carried out using samples taken from Boreholes R-1, R-2, and R-3. Six CK o UC tests were performed

on these samples using recompression (reconsolidated to the in-situ effective stress state) procedures

(Bjerrum 1973). Results obtained from these tests were compared with the strengths predicted using the

SHANSEP approach in Figure 38, and good agreement between these two types of approaches are

observed (Chin et al. 1994).

Undrained shear strengths measured using saturated unconsolidated undrained (SUU) shear test and

unsaturated unconsolidated undrained (UUU) shear tests on samples from K1 and K1/T2 zones were

compared with results derived from the normalized strength relationships and those tested using recom

pression procedures in Figure 39. As can be seen, the UUU and SUU test results are consistently lower,

and the results of SUU tests are particularly poor. The reason for this difference is simply that the effective

stresses under UUU tests are unknown because pore water

pressures are not measured, and consolidation

stress before SUU test shearing is only 5 kPa at MAA Laboratory (i.e. sample consolidated is under a

confining stress 5 kPa larger than the back pressure). It is obvious that the actual effective stresses in

UUU and SUU tests are significantly lower than those exist at site.

6 IN-SITU TESTS

In Taipei, one of the most widely used field tests in this region for soil properties characterization is

the standard penetration test (SPT). Other commonly used in-situ test methods include field vane shear

tests, cone penetration tests, pressuremeter tests. Geophysics probing techniques such as the electrical

resistivity method and downhole velocity logging method also have been used in many projects. Most

in-situ tests summarized in this section were conducted in the area adjacent to the Taipei City Hall Station 0 100 200 300 s u (kPa) from tests on samples reconsolidated to in-situ stresses 0 100 200 300 s u (k P a) e s t i m a t e d u s i n g S H A N S E P r e l a t i o n s h i p 1.00 0.85 s u (SHANSEP) s u (Recompression) =1.15 Symbol Borehole R1 R2 R3

Figure 38. Comparison of s u between recompression and SHANSEP type of tests (after Chin et al. 1994). 0 100 200 300 s u (kPa) 50 40 30 20 10 0 D e p t h (m) SUU UUU UUU test SUU test CK o UE test *(R1) CK o UC test *(R1) CK o UC test *(R2) CK o UC test *(R3) * Recompression to in-situ stress state

Figure 39. Comparison of s u from various test types (after Chin et al. 1994). SPT N-VALUE 0 5 10 15 20 25 30 35 40 45 0 5 10 15 20 25 30 D e p t h (m) BH-A BH-B BH-C BH-D BH-E

Figure 40. Typical SPT-N profile of K1 zone in Taipei.

of TRTS, which is located in K1 Zone. The typical profile of the site is represented by the profile of

Borehole R-1 shown in Figures 12 and 13.

6.1 Standard penetration test

Standard penetration tests are used very extensively in a routine site exploration program. It provides

data for design with the use of empirical correlations. In addition, disturbed samples can be obtained for

physical property tests in laboratory. In general, a great amount of physical property data often can be

obtained in a typical site exploration project. In Taiwan, SPT is carried out according to ASTM 1586.

However, it should be noted that brass liners are always used together with the split spoon barrels. After

samples were taken, they were sealed right the way and sent for subsequent laboratory tests. It is noted

that although many of these data may not be useful (e.g. water content of cohesionless soil obtained

from a split spoon sample), a lot of data on physical properties have been accumulated because of this

practice. They have become very useful once a systematic framework has been set up for geotechnical

zoning works. Statistics of physical properties shown in Tables 4 to 8 were able to be compiled as a

result of significant amounts of data collected. A typical SPT-N value profile for the K1 soil is shown in

Figure 40.

6.2 Field vane shear test

Field vane (FV) shear tests were used widely to obtain the in-situ undrained shear strength for Taipei

cohesive deposits. For Taipei Clay, tests conducted in several locations in close proximity within the K1

Zone were available, and they can be used to portrait the s_u profile of the clay, as shown in Figure 41.

The undisturbed and remolded strength values are shown in

Figure 41a, where the undrained strength

corrected using Bjerrum's factors, μ , (Bjerrum 1972) also is shown in the figure. As can be seen, no appreciable difference can be depicted between the corrected and uncorrected peak FV strength for this clay. For comparison, the s_u profile based on the direct simple shear SHANSEP equation (Equation 6) also is plotted in Figure 41a. It can be seen that the trend of the s_u (FV) profile is quite consistent with the s_u (DSS) profile, but the value of s_u (FV) is somewhat smaller than that of s_u (DSS). On average, the s_u (DSS)/ s_u (FV) is about 1.5 for normally consolidated to slightly overconsolidated Layer IV Taipei Clay. Using the state profile for K1 soil shown in Figure 10, values of s_u (FV)/ σ'_{vp} can be calculated, and the computed value is about 0.13 on average. This is much smaller than that suggested by Larsson (1980), where it is suggested that the s_u (FV)/ σ'_{vp} ratio be likely about 0.23 ± 0.04 . The sensitivity of the clay is shown in Figure 41b, and the value is about 3.0 on average. In another site investigation report (MAA 1983), results of unconfined compression strength tests on K1 Clay showed that the sensitivity ranges from 4 to 8 with an average of 5.9. In that study (MAA 1983), it also was found that the FV sensitivity is always lower (smaller than 4.0).

Figure 41. Field vane shear s_u profile for Taipei Clay.

appreciable difference can be depicted between the corrected and uncorrected peak FV strength for this

clay. For comparison, the s_u profile based on the direct simple shear SHANSEP equation (Equation 6)

also is plotted in Figure 41a. It can be seen that the trend of the s_u (FV) profile is quite consistent with

the s_u (DSS) profile, but the value of s_u (FV) is somewhat smaller than that of s_u (DSS). On average, the

s_u (DSS)/ s_u (FV) is about 1.5 for normally consolidated to slightly overconsolidated Layer IV Taipei

Clay. Using the state profile for K1 soil shown in Figure 10, values of s_u (FV)/ σ'_{vp} can be calculated,

and the computed value is about 0.13 on average. This is much smaller than that suggested by Larsson

(1980), where it is suggested that the s_u (FV)/ σ'_{vp} ratio be likely about 0.23 ± 0.04 . The sensitivity of the clay is shown in Figure 41b, and the value is about 3.0 on average. In another site

investigation report (MAA 1983), results of unconfined compression strength tests on K1 Clay showed

that the sensitivity ranges from 4 to 8 with an average of 5.9. In that study (MAA 1983), it also was

found that the FV sensitivity is always lower (smaller than 4.0).

6.3 Cone/Piezocone penetration test

The electric cone penetration test was generally conducted

in accordance with ASTM standard D3441.

During the development of the initial network of the TRTS, the electric cone penetration tests were

carried out extensively (Wong et al. 1993). Cones equipped with pore pressure transducer to measure

the excess pore pressure developed while conducting the test are known as piezocone, and the enhanced

procedure is known as the piezocone test. In Taipei, the piezocone test is becoming increasingly popular

for practicing engineers in characterizing soils in recent years. General applications of this test include

detecting soil variability, classifying soil, developing OCR profile, finding undrained shear strength,

estimating consolidation coefficient and permeability, and more. Typical result of a piezocone test on

K1 clay is illustrated by Figure 42 based on a set of data retrieved from a site near the Sungshan Airport

in which the cone tip resistance, Q_t , local sleeve friction, F_s , friction ratio in terms of F_s/Q_t and pore

pressure are plotted versus depth. One of the methods for determining undrained shear strength is presented herein as an example to

demonstrate the application of the CPT. In regional practice, the s_u of a clayey soil can be determined 1.5
Tip resistance Q_t (MPa) 0 5 10 15 20 25 30 35 40 45 0 5
10 15 20 25 30 35 40 45 0 5 10 15 20 25 30 35 40 45 0 5 10
15 25 30 35 40 45 Friction ratio F_s/Q_t (%) 0 80.10 Local
friction F_s (MPa) 0 20 D e p t h (m) Pore pressure P_w (MPa) 05

Figure 42. Typical CPT results for Taipei Clay.

by substituting the measured cone tip resistance, q_c , the total overburden stress, σ_{vo} , and the cone factor

N_k at given depths into the following equation:

The value of N_k is a result of calibration based on

available CPT data and s_u value measured from

DSS tests conducted at given depths. For normally consolidated and slightly overconsolidated K1 clay,

the N_k value is approximately 8.5 on average.

A variety of soil parameters can be determined from the CPT. Using this method, the ground variability

can be detected more effectively compared to most of the other approaches because the data are retrieved

continuously during the test. However, as being mentioned, the early CPT devices tended to have lower

than expected system rigidity which often resulted in unwanted errors. In addition, it has been found

that the deairing process for the porous material in the device was sometimes not as good as expected. It

is probably due to operator's negligence or unfamiliarity with the test procedure. As reported by Wong

et al. (1993), this can cause lag of the piezocone response to the pore pressure change and result in

ineffectiveness in detecting thin layers. Figure 43 shows a case history in which a typical response

of the piezocone to the pore pressure measured by poorly saturated device was shown in part (a) and

the response measured by a better saturated device was given in part (b). As can be seen, the measured

response in Figure 43a is almost meaningless whereas apparent indication of change in ground properties

and pore pressures can be identified in Figure 43b.

6.4 Pressuremeter test

The pressuremeter test often is called the lateral load test (LLT) due to local preference. Commercially

available testing equipment include Menard pressuremeter from France, OYO's K-value tester, LLT and

KKT from Japan. One of the commonly used apparatus in local practice is the LLT device made by OYO

Corporation of Japan. This test method is generally performed to evaluate the deformation parameters

of the subsoils such as the coefficient of horizontal subgrade reaction, coefficient of earth pressure at

rest, shear modulus and elastic modulus. For K1 clay, a typical LLT result is shown in Figure 44. The

LLT results also can be used to estimate the undrained shear strength using the following equation:

Figure 43. Effects of saturation of filter tip on CPT results. 4.0 4.5 5.0 5.5 6.0 6.5 7.0 Radius(cm) P r e s s u r e , P e (k P a) 0.0 1.0 2.0 3.0 4.0 5.0 H, cm P o P y ' P L ' 400 300 200 0 100

Figure 44. Typical LLT results of Taipei Clay.

in which p_L = LLT limit stress, p_o = LLT total horizontal stress, and N_q = LLT factor. Values of N_q for

Taipei Clay have been evaluated with reference to the s_u measured from DSS tests. It was found that for

Taipei Clay the N_q value is about 4.4 based on seven sets of available data. Modulus and K_o can also be

derived from LLT. It has to be noted that results for K1 clay are much smaller than the typical values

reported by others (e.g. Kulhawy & Mayne 1990). The reason can probably be attributed to the amount

of significant disturbance. Shear wave velocity, V_s (m/s)
0 10 20 30 40 50 60 70 80 0 200 400 600 D e p t h (m)
TRTS Xinyi line Taipei 101 A Taipei 101 B Shear wave velocity

Figure 45. Shear wave velocity profile for Taipei Clay.

6.5 Shear wave velocity test

The most common seismic methods in the context of geotechnical engineering used in Taipei region

perhaps is the down-hole velocity logging. This method

consists of generating shock waves in the ground, allowing them to propagate through soil, then using sensors in a predrilled borehole to measure the wave velocity as they arrive at specified depths. Appropriate velocity-depth functions and direct correlation between seismic reflections and depth can be obtained from this method. Figure 45 shows a typical shear wave velocity profile in the K1 Zone. This plot was established based on test results conducted at the construction sites of Taipei 101 and TRTS Xinyi Line.

6.6 Electrical resistivity method

Electrical resistivity method is one of the electrical prospecting. This method has been used routinely in TRTS projects for the purpose of obtaining parameters for electricity system design. The objective of this method is to identify the structure of the resistivity layers based on the apparent ground resistivity. A typical plot of vertical electrical resistivity sounding and resistivity log is illustrated herein by one of the tests conducted at T2 Zone as shown in Figure 46. Although this is one of the approaches that often used to investigate ground profile composition, it is very sensitive to environmental interference and operator's craftsmanship.

7 ENGINEERING PROBLEMS

Due to the high-density urban development in past decades and the decreasing availability of construction space, more and more underground construction projects such as cut-and-cover excavations and tunneling have to extend their depths to Layer IV or deeper layers of Sungshan Formation in recent years. This

brought great concerns to the potential of differential settlement, which is often a result of underground

construction activities and can be detrimental to the structure of the buildings being influenced. Impact

of new construction project to adjacent structures has to be taken into consideration during design and

construction.

With the high sensitivity and the closer to or higher than the liquid limit natural water content of the

Layer IV clay, construction activities often result in shear strength reduction as the ground is subject to 1 10
100 Half spacing (m) 1 10 100 A p p a r e n t r e s i s t i
v i t y (O h m m) Depth (m) Resistivity (Ohm-m) 0-2 34
2-6 6 6-21 24.6 21-50 24.8

Figure 46. Typical results of vertical electrical resistivity sounding curve and resistivity log for Taipei Clay.

disturbance. For cut-and-cover construction, to ensure the stability of soil below the excavation surface

and the safety of the infrastructures nearby, the retaining structures need to penetrate into a depth

significantly greater than the final excavation depth. Because the mobilized passive earth pressure is

usually lower than that expected, large lateral displacement of the retaining walls will occur and associate

with significant amount of ground settlement. To restrain this lateral displacement, prevent basal heave

and mitigate the potential of differential settlement on the ground outside the excavation zone, high

stiffness and elongated retaining walls, prestressed bracing struts, barrette, cross panel, and ground

improvement measures such as high pressure jet grouting for soils outside the excavation zone and

below the excavation level usually were taken. Another

example demonstrated herein is tunneling. The ground settlement over tunnels due to tunnel

ing was primarily the results of the following phased construction activities including (a) shield machine

face advancement, over-cutting and shield advancing, (b) the closure of tail void between the shield

machine and surrounding soil, and (c) consolidation. Hwang, et al. (1995) combined the phases (a) and

(b) as "ground loss settlement" and singled out consolidation settlement from others. For soft clays, the

consolidation settlement may be a predominant source of settlements and may drag on for days, if not

months. Similar to deep excavation aforementioned, ground settlement resulted from tunneling can be

detrimental if the ground behavior was not appropriately evaluated beforehand. Prior to the initiating of the TRTS project, experience consisting of empiricism and precedent seemed

to play a lopsided role in general geotechnical engineering practice in the region of Taipei. With increasing

demands to tackle project under complex construction conditions and resolve the engineering problems

that engineers must face, in-depth understanding of soil behavior became a critical issue. Nowadays the

empirical or semi-empirical procedures still plays an important role in geotechnical engineering practice,

other approaches such as physical modeling, analytical modeling, or numerical modeling based design

gradually gain their popularity and importance. It probably can be attributed to the state-of-art mechanical

and electrical field and laboratory testing equipment and the fast developed computer technologies which

in turn make the once seemingly difficult or impossible task an easy one now. One of the most powerful tools for solving today's engineering problems is the joint

application of

realistic soil models and advanced numerical modeling in design practice. An example given herein

is related to a soil-structure interaction problem. Two bored tunnels planned to be excavated by shield

machine will go across an existing cut-and-cover twin box tunnel from underneath. One of the twin box

tunnel consists of the Taiwan High Speed Rail tracks and the Taiwan Railway tracks are in another. The

bored tunnels are part of the new Sungshan Line of the TRTS near the Taipei Main Station (Chao et al.

2005). As far as the engineer is concerned, interruption of the regular operation of the High Speed Rail

by any of the adverse construction effects is the risk that can not be affordable. There was no precedent

case available and thus empirical or semi-empirical design method was not an option. The 2-D simulation

approach was inappropriate herein because the potential influence of the construction activities to the box

tunnels was 3-D in essence as can be seen in Figure 47. Thus, Class-A 3-D evaluations were conducted

Figure 47. Proposed construction of new Sungshan Line of the TRTS (after Chao et al. 2005).

Table 10. Evaluated MIT-S1 parameters for Taipei Clay (Pestana & Whittle 1999, Hsieh 2006). Boston Lower blue Taipei cromer

Test Type	Parameter	Physical contribution/meaning	clay	Clay	till
-----------	-----------	-------------------------------	------	------	------

1-D	p c	Compressibility of limiting	0.178	0.16	0.167
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Consolidation compression Curve (LCC)

(Oedometer

CRS ,K	θ	-triaxial)	D	Non-linear volumetric	0.04	0.01	0.04
r	Swelling behavior	0.85	0.8	0.8	h	Irrecoverable plastic	

strain 6 15 6

K_0 -triaxial K_0 for NC clay 0.49 0.52 0.50

K_0 -triaxial μ' Poisson's ratio at stress reversal 0.24
0.21 0.225

OCR = 5-10 controlling $2G_{max}/K_{max}$ Non-linear Poisson's
ratio. 1 1 0.6 Stress path in 1-D unloading

Undrained ϕ' cs Critical state friction angle in triaxial
33.5 ° 30 ° 30.0 °

triaxial compression

shear tests:

CK 0 UC ϕ' m Geometry of bounding surface 46 ° 35 ° 42.0 °

(OCR=1) Stress path of undrained CK 0 UTC/TE

CK 0 UE m tests 0.8 1.4 0.65

(OCR=1)

CK 0 UC ω s Non-linear at small strains in 8 12 8.0

(OCR=2) undrained shear

Resonant column or C b Small strain stiffness at load
reversal 450 450 450

shear wave velocity

CK 0 UC ψ Rate of evolution of anisotropy 15 8 20

CK 0 UE (rotation of bounding surface)

in the design phase to obtain insight into various
soil-structure interaction behaviors associated with the

complicated nature.

To deal with complexity, an advanced numerical modeling
using MIT-E3 model was established for

Taipei Clay at the same time the initial network of the
TRTS was implemented. Without providing field

monitored information prior to the modeling, satisfactory

prediction results were obtained for the deep

excavation of the Convention Center of Taipei World Trade Center (Whittle et al. 1993). More recently,

an updated and unified constitutive model for both sand and clay, namely MIT-S1, was developed by

Pestana and Whittle (1999). Significant amount of endeavors have been put into it for evaluating the

parameters and assessing the effectiveness of this model for Taipei Clay.

Table 10 tabulated the results of the parametric evaluation for Taipei Clay. For comparison, parameters

for Boston Blue Clay and Lower Cromer Till also are provided. Table 11 summarized the results of

Table 11. Comparison of measured and predicted undrained shear strength ratios for Taipei Clay. MIT-S1 Measured MIT-S1 Measured MIT-S1 Measured

OCR 1 1 2 2 4 4

K o 0.52-0.55 0.52-0.55 0.7-0.77 0.7-0.77 0.9-1.06 0.9-1.06

s uTC / σ'_{vc} (1) 0.30 0.29 0.54 0.51 0.86 0.83

s uTE / σ'_{vc} (1) 0.18 0.19 0.32 0.33 0.60 0.63

s uPSA / σ'_{vc} 0.32 - 0.55 - 0.90 -

s uPSP / σ'_{vc} 0.24 - 0.42 - 0.73 -

s uDSS / σ'_{vc} (1) 0.23 - - 0.2 0.39 0.65

s uDSS / σ'_{vc} (2) 0.23 0.39 0.67

Note: measured values for (1) from Whittle et al. (1993); (2) from Liu (1999).

comparison between measured and predicted values of various modes of undrained shear strengths at a

variety of OCR values based on the evaluated parameters. As can be seen, results of the elementary level

evaluation showed that the MIT-S1 model was capable of

providing excellent predictions on the undrained

shear strength ratios for Taipei Clay. Thus, it is very encouraging to carry on with further investigation to

evaluate the prototype engineering problems. Nevertheless, engineers have to bear in mind that the MIT

S1 model is a rate independent model. Further studies on rate dependent effect for Taipei Clay are required.

8 SUMMARY AND CONCLUSIONS

Taipei City is located in the northern part of Taiwan and occupies more than half of the Taipei Basin. More

and more building and infrastructure developments with increasing scale and significance are built. As a

result, extensive investigations have been conducted on the soil deposits of Taipei, especially the cohesive

deposits. This paper presents a comprehensive overview of behaviors and properties of Taipei Clay. The Taipei Basin was formed as a result of reverse and normal faulting in northern Taiwan. The main

soil deposit of the Basin is Sungshan Formation. The late Pleistocene to Holocene Sungshan Formation in

the Taipei Basin was deposited under the fluvial, lacustrine, estuary and brackish water bay environments.

The maximum thickness of Sungshan Formation is about 120 m. The greatest thickness of Sungshan

Formation is in the northwestern Wuku area, and it gradually thins toward the southeast and the northeast.

Near the end of deposition of the formation, the brackish water gradually retreated from the basin. Since

Taipei region subsided around 0.3–0.4 Ma and the Taipei Basin has been formed, the area has undergone

a continuous sedimentation process, probably without recognizable erosion. However, due to excessive

water pumping from the underlying Chingmei Formation, groundwater pressure of Sungshan Formation

have been lower and ground subsidence was resulted. Even though the water pumping is now prohibited

by the government, it is very important that the groundwater pressure be continuously monitored. A major step toward systematic understandings of the Taipei soils was achieved by the first city-wide

geotechnical mapping conducted and completed in 1986. The city was divided into six geotechnical zones

based on a very scrupulous review of physical properties of soil samples and the depositional environment

of three rivers flowing into the Basin. The results had great implications for subsequent researches as

well as engineering practices. Results of zoning and the associated statistical characterization of Taipei

Clay were presented. This geotechnical zoning work should be further extended to cover the whole Basin

with the same level of scrutiny as it was developed in the mid-1980's. Engineering characteristics of Taipei Clay have been investigated for decades. A more comprehensive

understanding was achieved through many major construction projects in the late 1980's and early 1990's.

The compressibility of Taipei Clay has been studied both in CRS and oedometer tests. The CR value for

Taipei Clay obtained from the CRS test is about 0.18, which is very comparable to that of the Boston

Blue Clay (CR = 0.17). Permeability characteristics of Taipei Clay has not been extensively studied.

A lot of efforts have been put on the study of strengths of Taipei Clay. Comparisons have been made

between undrained shear strength commonly used in local practice with more controlled tests. SHANSEP

undrained shear strength ratios are established for CKoUTC, CKoUDSS, and CKoUE modes of shearing.

These results are also comparable to those of Boston Blue

Clay. To study these engineering characteristics, most samples were taken from construction projects.

Though high quality low disturbance samples were always required, not many investigations on dis

turbance effect have been conducted. A preliminary study reported by Hu et al. (2005), based on the

concept of "Specimen Quality Designation" (SQD), showed that reasonably good undisturbed samples

can be obtained following procedures used in the Designated Task. Investigation on the disturbance effect

should be further studied in the future. It is noted that many test results presented herein are in general

accordance with ASTM or other standards, yet rate effect is very important and deserves more atten

tion. In addition, resedimentation technique should also be considered in investigation of engineering

characteristic of Taipei Clay.

Various in-situ tests have been used in Taipei. However, some results have not been rationally inter

preted in practice. Local correlations are not always available. More systematic review would be needed

for further applications of in-situ tests in Taipei. More thorough theoretical validation (e.g. use of the

strain path method) or calibration chamber tests are needed to establish local correlations. Nevertheless,

based on a first order calibration, it was found that VST, CPT, and LLT can be used to estimate undrained

shear strength fairly consistently. Using s_u (DSS) as the reference, it is found that the cone factor N_k is

about 8.5 and the LLT factor is about 4.4.

In this paper, model predictions on single element level have been discussed. It is very encouraging

that the MIT-S1 model gives very reasonable predictions on

undrained shear strength for various modes

of shearing at various OCRs. For tackling very complicated construction projects, advanced numerical

modeling may require further enhancement of the available constitutive models. Small strain behavior

of soil is important for accurate modeling, and only limited studies have been done for Taipei Clay (e.g.

Kung 2003). For comprehensive modeling, the rate dependent behavior is also important and should

be modeled correctly. Liu (1999) has initiated the study of rate dependent behavior of Taipei Clay, but

more efforts on this subject are needed in the future. It will be a very exciting and challenging task to

model the whole spectrum of soil deposits of the Taipei Basin, from silty sand to silty clay, in a unified

framework expressed in a single set of parameters.

9 ACKNOWLEDGEMENTS

First of all the Authors would like to thank Dr. Za-Chieh Moh. He initiated the systematic study of Taipei

soils and promoted the geometrical zoning of the Taipei Basin. Dr. Moh encouraged and guided the

Authors to pursue a better understanding of Taipei Clay. The Authors are very grateful to Dr. Jui-Tun

Chou for his valuable input on the geology of Taipei. The Authors are also very grateful to Prof. Ming

Lang Lin for his advice on geology and its relationship with the engineering properties of Taipei soils.

The Authors want to thank Dr. Chuan-Chi Liu for providing valuable information and test data of Taipei

Clay. The Authors especially appreciate a long-time working relationship with Dr. Liu in the study of

Taipei soils. Prof. Yo-Ming Hsieh is greatly appreciated for providing useful comments and suggestions

on soil modeling and comparisons between Taipei Clay and Boston Blue Clay. The Authors also thank

Prof. Liang-Chien Chen and Dr Jiann-Yeou Rau of Center for Space and Remote Sensing Research,

National Central University for providing satellite image of Taipei. Colleagues at MAA, Ms. Hsiao-Han

Wu, Mr. Ching-Hui Huang, Mr. Hsiao-Chin Tseng, Mr. Wei-Jun Chung, Dr. Sung-Ju Chan, Dr. Jung

Feng Chang, Mr. Chung-Ren Chou, Mr. Cheng-Chen Wu, Mr. Pei-Lin Hou, and Dr. Tian Ho Seah, help

prepare this manuscript. Their support and assistance are very much appreciated. Appreciation is also

due to the Organizing Committee, Prof. Tan Thiam Soon and Prof. Phoon Kok Kwang, for inviting the

Authors to present this paper.

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of Taipei Basin: 198-202. (in Chinese) Characterisation and

Characterization of alluvial deposits in Mekong Delta

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ABSTRACT: Soil investigations of soft alluvial soils in
Mekong River Delta were carried out at two

sites, Can Tho and Tan An, South Vietnam. The Can Tho site
is beside the levee of Hau River, one of

the major streams of Mekong with very large discharge rate
and the Tan An site is beside Bam Co Tay

River, a meandering river with relatively lower discharge
rate. The differences of depositional environ

ments create different types of soil profiles at the two
sites, relatively low plastic soils with significant

heterogeneity in the vertical direction at the Can Tho site
and high plastic uniform clay at the Tan An site.

In the investigation, soil samplings with Japanese type
thin-walled tube sampler with fixed piston (JFP)

and Shelby tube samplers were conducted. Consolidation and
strength properties of the clays retrieved

by the two samplers were studied. Various in-situ tests
were also conducted at the site, such as piezo

cone penetration test, Marchetti's flat dilatometer test, a
field vane test. Although carbon isotope dating

showed the two clays to be very recent (less than 6300 BP
at $z = 7.0$ m for the Tan An clay and 9500 BP

at $z = 6.5$ m for the Can Tho clay), the mechanical and
physical properties are quite different. Both clays

have developed high degree of microstructures, which has been confirmed from CRS tests of very good

quality samples. The effects of sampling disturbance on the engineering mechanical properties of the

samples retrieved by the two samplers are also studied.

1 INTRODUCTION

Mekong Delta is one of the largest deltas in Asia, which has a huge potential for various industries,

especially agriculture. This delta has been formed since Tertiary, but majority of the current delta is

covered by Holocene deposit transported by Mekong River after the last glacier (Huu Chiem, 1993).

Average water discharge of the river is 1.5×10^4 m³/s and average annual suspended load is 1.6×10^8 t

(Hori and Saito, 2003). This Holocene deposit created in a very short geological time shows very unique

feature. Figure 1 is the distribution of soil status at shallow depth in Mekong Delta (At lat, 2002), which

clearly shows the geological history and current natural conditions. Acid-sulfate soil is evidence that the

delta was under the sea in transgression period (5000-6000 BP). Fe³⁺ and SO₄²⁻ in the sea water formed

pyrite (FeS₂) in the deposit with reduction condition under the mangrove forest. But this marine deposit

became land by regression and oxidization of pyrite produced acid sulfate, resulting in acid-sulfate soils

widely prevailing in the delta.

As the average elevation of the Mekong Delta is less than one meter, the delta has suffered flooding

and salt intrusion for long periods of time. During rainy season, huge area more than a million hectares

along the major rivers in the delta is flooded. The soils are transported by the river and sedimented along

the river forming fluvial deposits. On the other hand, large tidal range of South China Sea causes salt

intrusion, which creates saline soils in the coastal area as shown in Figure 1. According to Hori and Saito

(2003), the current Mekong Delta is characterized as mixed energy type delta, of which progression

is controlled by wave and tide. But in the beginning of the delta formation, it was supposed to be

tide-dominated type delta.

Figure 1. Distribution of soil status in Mekong Delta (At lat, 2002). From the above mentioned dynamics of Mekong Delta, it can be expected that the formation of the

alluvial deposit in the delta has been affected by various factors, flooding, tidal and wave energy, as well

as the discharge of the soils and eustacy. This might create quite complicated subsoil conditions. Man (2003) reviewed more than thirty soil investigation reports on soft clayey soils in Mekong Delta.

Figure 2 is the soil profiles at various location of Mekong Delta given in his report, showing very com

plex and quite deep soft alluvial soils not only in the coastal area but also almost all area in the delta.

From the soil investigation reports, Man (2003) compiled the consolidation properties of the soft clayey

soils in the delta and showed curious trends in their properties. Figure 3 is one example where overcon

solidation ratios (OCRs) decrease with depth and most OCRs are less than unity. From the change of

void ratio by recompression to the in-situ effective vertical stress (Figure 4), he attributed the low sample

quality to the strange mechanical properties of the soils given in the reports. Disturbance of the sample

might be caused by sampling and other processes after sampling. Sampling methods commonly used in

Vietnam for relatively deep soft soils is open sampling with Shelby tube, although many literatures have

shown the advantage of fixed piston type sampler in the sampling quality compared with the open type

sampler especially for soft soils (e.g., Marcuson & Franklin, 1979, Tanaka and Tanaka, 1999). Because

of such disturbance commonly observed in the soft clay samples as well as the complicated formation

process of alluvial deposit in the delta, mechanical properties of Mekong Delta clay have not been well

studied yet. In order to provide more reliable geotechnical information in Mekong Delta soft alluvial soils, col

laboration researches on characterization of soft Mekong Delta clay have been done at two sites, Can

Tho and Tan An between Japanese partners (Tokyo Institute of Technology (TIT) and Port and Air

port Research Institute (PARI)) and Vietnamese partners (Ho Chi Minh City University of Technology

(HCUT), Transport Engineering Design Incorporate (TEDI) and Union Survey Cooperation (USCo),

Asian Institute of Technology (AIT)). In this collaborative research, clay samplings with Japanese type

thin-walled tube sampler with fixed piston (JFP) and Shelby tube samplers were conducted at the two

sites. Consolidation and strength properties of the clay sampled by the two methods were obtained from

the laboratories in PARI, TEDI and HCUT using the common procedures and equipment in Japan and

Vietnam and the test results obtained in Japan and Vietnam were compared. Various in-situ tests were

also conducted at the site, such as piezocone penetration test, Marchetti's flat dilatometer test, field vane

test. This report describes the results of the field tests

and the laboratory tests done at PARI. Chaâu Phuû CaûiNôôüc
BaïcLieâu Town Thaïnh GioàngTroâm Cao Laõnh S o u t h C h i
n a O c e a n CaiLaây BìnhHồng Phuû Phong MôûCaøy CôøÑôû
Bridge (Ôa Moân) Quang Trung Bridge DöôngHoøa BìnhThaïnh
Legend

Figure 2. Soil stratigraphy of some boreholes in Mekong
delta (after Man, 2003). 0 5 10 15 20 25 30 35 40 0.1 1 10
Log OCR D e p t h (m) (1) (2) (3) (4) (5) (6) (7) (8) (9)
(10) (12) (13) (14) (15) (16) (17) (18) (22) (23)

Figure 3. Overconsolidation ratios reported on clay samples
at various sites in Mekong Delta (after Man, 2003). 0 2 43
0.0 0.1 0.2 0.3 1

C h

a n

g e

o

f v

o i

d

r a

t i o

Δ

e /

e 0 OCR (1) (2) (3) (4) (5) (6) (7) (8) (9) (10) (12) (13)
(14) (15) (16) (17) (18) (22) (23) Good to fair Good to
fair Poor Very poor Poor Very poor Very good to excellent
Very good to excellent

Figure 4. Changes of void ratio due to recompression to the
effective overburden stress obtained in oedometer tests

on clay samples at various sites in Mekong Delta (after
Man, 2003). Tan An CAMBODIA VIETNAM Long Xuyen My Tho Can
Tho Ho Chi Minh City H a u R i v e r T i e n R i v e r S o
u t h C h i n a S e a T h a i l a n d B a y V a m C o T a y
R .

Figure 5. Location of Tan An and Can Tho.

2 TEST SITES AND SOIL PROFILES

2.1 Tan An site

Tan An is located about 50 km southwest from Ho Chi Minh City (Figure 5). As seen in Figure 1, the

saline soils indicates the effect of salt intrusion in this area, although the Tan An is more than 30 km

from the coastal line. The test site is a swamp area beside Bam Co Tay River. As Bam Co Tay River is

not part of Mekong River. Meandering shape suggested the relatively low discharge and sedimentation

of fine particles on the riverside. 0 20 PI 2.6 2.7 2.8 p s
(g/cm³) water contents & PI (%) 0 80 Grain size
distribution (%) Liquidity index LI (%) under 2µm 5µm 75µm
w P 20 40 60 100 40 60 80 100 120 0 0.25 0.5 0.75 1

2.0

3.0

4.0

5.0

6.0

D

e p

t h

z

(m

)

7.0

8.0 w n w L

Figure 6. Profile of physical properties at Tan An site. 0 5 10 15 20 0 u 0 4000 0 250 500 750 0 500 1000 0 q t (kPa) q t (kPa) u 2 (kPa) f s (kPa) 200 0 10 20 30 40 50 f s (kPa)

D

e p

t h

z

(m

) CPT CPT 1 2

Figure 7. Results of CPTU at Can Tho site.

Figures 6 and 7 show the profile of physical properties and results of piezocone test (CPTU) results

at the Tan An site respectively. At the Tan An site, 1m thick fill material was placed on very soft clay of

about 6.5 m thickness. As the stiffness of the layer underlying the soft clay was quite high (see Figure 7),

the sampling tubes could not be inserted using the sampling equipment employed at the Tan An site.

2.2 Can Tho site

As seen in Figure 5, Can Tho City is about 160 km southwest from Ho Chi Minh City situated at the

right bank of Hau River, one of the major streams of Mekong River in Vietnam. The field tests were

carried out at a working yard for the construction of Can Tho Bridge, a cable stay bridge crossing Hau Water content w (%) Liquidity index LI pH p s (g/cm 3) grain size distribution(%) 12106 82 4 2.01.51.00.50.0 806040200100802.80 0 20 40 602.60 2.65 2.70 2.75 under 2µm 5µm 75µm w P w L w n 0 5 10 15 20 25 30 D e p t h z (m)

Figure 8. Profile of physical properties and pH at Can Tho site. 0 5 10 15 u 0 0 1000 2000 0 200 400 600 800 0 10 20 30 f s (kPa) u 2 (kPa)q t (kPa) D e p t h z (m) 30 25 20

Figure 9. Results of CPTU at Can Tho site.

River. The working yard for the tests was the left bank of the river, which is on the opposite side from

Can Tho City. The site was originally flooding area beside the levee. The elevation of the site was raised

from 1.4 m to 2.3 m by filling, but it subsided to about 2 m due to consolidation. Although the location is

about 100 km upstream from the river mouth, the river water level oscillates daily due to the large tidal

range of South China Sea. But large discharge of the Mekong Rivers prevents the saline intrusion along

the major river sides and the ample supply of fresh water including flood has created the better ground

condition for agriculture in the river side area than the surrounding area with acidic soils (see Figure 1). Profiles of physical properties and CPTU results are shown in Figures 8 and 9, respectively. At the

Can Tho site, sampling was done to the depth of 26 m which was the maximum depth that JFP sampler

can be inserted by the water pump used. From the trend of cone resistance (q_t), and pore water pressure

measured behind the cone tip (u_2), both of which increases with depth, the deposit is considered typical

normally consolidated soft clay. But sudden increases of q_t and drops of u_2 from the trends indicate very

shallow sand layer or intermediate soils with high permeability. From carbon isotope C 14 dating shown in Table 1, it can be said that all soils investigated at the Can Tho

site can be categorized as Holocene deposits. Although the dated year may not give the correct deposition

Table 1. Results of carbon isotope C 14 dating.

Tan An site Can Tho site

Depth (m) Year (BP) dz/dt (mm/y) Depth (m) Year (BP) dz/dt

(mm/y)

4.0 1545 ± 40 1.9 5.8 3739 ± 30 1.3

7.0 6346 ± 35 0.6 13.8 4100 ± 30 22.2 19.7 6385 ± 30 2.6
23.7 8146 ± 35 2.3 25.6 9498 ± 35 1.4

Table 2. In-situ tests conducted.

Type of test Can Tho site Tan An site

Piezocene penetration test (CPTU) Yes Yes

Marchetti's dilatometer test (DMT) Yes Yes

Field vane test (FV) Yes No

Hydraulic fracturing test (HFT) No Yes

Multi-point spectra analysis of surface wave (SASW) No Yes

Table 3. Laboratory tests conducted at PARI. Can Tho clay
Tan An clay

Type of set JFP Shelby JFP Shelby

Index test Yes No Yes Yes

pH measurement Yes Yes No No

Incremental loading oedometer test Yes Yes Yes Yes

CRS consolidation test Yes Yes Yes Yes

Unconfined test with suction measurement Yes Yes Yes Yes

Constant volume direct shear test (CVDST) Yes No No No

CK 0 UC and CK 0 UE triaxial tests Yes No No No

Laboratory vane test Yes Yes No No

period, the deposition speed in terms of the current
thickness is at least more than 2.6 mm/year, which

is much higher than that of Tan An clay (1.0 mm/y). From
this dating and the CPTU results (Figure 9) it

can be expected that a rapid deposition took place at the
depth from 5 m to 17 m.

3 TEST PROGRAMS

3.1 In-situ tests and samplings

In-situ tests conducted at the two sites are listed in Table 2. At both sites, soft soils were retrieved by

two sampling methods, i.e., Japanese type thin-walled tube sampler with fixed piston (JFP) and Shelby

tube samplers. At Tan An site, both types of sampling were conducted from 3.5 m to 7.5 m every 1 m.

At Can Tho site, JFP was conducted to 26 m depth, every 1 m from 3 m to 16 m and every 2 m from 16

to 26 m. Shelby tube sampling was done for the shallower depth. Originally Shelby tube sampling was

planned from 5 m to 9 m with 1 m interval, however the open type sampler could not retrieve the soil at

some depths. Therefore the Shelby tube sampling was continued to 13 m depth to obtain five samples.

3.2 Laboratory tests

Laboratory tests done at PARI for the clay samples collected by Japanese type fixed piston thin wall

sampler (JFP) and Shelby tube sampler at the two sites are listed in Table 3. As described in the subse

quent section, there were many holes in the samples of Tan An clay, which made it difficult to measure

Figure 10. evidences of bioturbations in Tan An clay: (a) holes probably made by micro-organism; (b) plants; (c)

evidence of organism.

Figure 11. Scanning electron micrographs of Tan An clay $z = 6.5$ m.

excess pore-water pressure, K_0 consolidated undrained shearing tests and constant volume direct shear

tests were not conducted for Tan An clay. Incremental loading oedometer tests of reconstituted samples

were also carried out to obtain the intrinsic compression behavior of the two clays.

4 MATERIAL COMPOSITIONS AND STRUCTURES

4.1 Grain size distributions

As seen in Figures 6 and 8, grain size distributions of Tan An and Can Tho sites are very different. Tan An

clay has large fraction of fine grain (under 2 μm) of more than 50% with relatively uniform distribution

in the sampling depths from 3.5 m to 7.5 m. While for Can Tho clay, the grain sizes are larger than

those of Tan An clay. The grain size distribution fluctuates at the shallower depth down to 16 m, which

corresponds to the variation of q_t and u_2 measured by CPTU (see Figure 9). It should be noted that some

local variation of soil profiles estimated by CPTU, e.g., $z = 15\text{--}17$ m, cannot be captured by the grain

size analyses and index tests done with the samples collected at 1 m depth interval above 16 m and 2 m

depth interval below 16 m.

4.2 Structures

4.2.1 Macrofabric and microfabric of Tan An clay

Due to very recent and shallow deposition, the Tan An clay shows some evidences of bioturbation

as shown in Figures 10. The most specific feature of the Tan An clay is randomly distributed holes,

which have been probably burrowed by micro organisms. Although the Tan An clay has quite uniform

microfabric with very thin particles with large pores (see Figure 11) and uniform composition in the

depth investigated (see Figure 6), results of CPTU are very sporadic and less repeatable. This can be

attributed to the holes in the clay.

Figure 12. Very thin sand seams in Can Tho clay.

Figure 13. Scanning electron micrographs of Can Tho clay:
(a) $z = 12.5$ m; (b) $z = 21.5$ m.

4.2.2 Macrofabric and microfabric of Can Tho clay

As seen in Figures 8 and 9, the Can Tho clay shows strong heterogeneity with depth. Clay dominates

the composition but very thin sand seams are occasionally included in the clay as shown in Figures 12.

The thickness and composition of sand seams or intermediate soils depends on the level of flooding and

discharge rate of Hau river. Figure 13(a) shows relatively larger clay particles and smaller pore size of

Can Tho clay than Tan An clay. From SEM image of the clay at depth of 21.5 m (Figure 13(b)), pyrite

can be observed, which suggests that the clay at this depth sedimented in marine environment. This can

also attributes to the relatively high acidity of the clay at deeper depth shown in Figure 8.

5 STATE AND INDEX PROPERTIES

5.1 Atterberg limits and activity

At the Tan An site, Atterberg limits and plasticity indices are quite uniform in the very soft clay layer

as seen in Figure 6. From the plasticity chart on various clay in the world (Figure 14), the Tan An clay

can be categorized as high plastic clay with $w_L = 100-120\%$ and $PI = 72-90$, well above A-line in the

chart. As for the Can Tho clay, the data points are also well above A-line, but w_L and PI range 44-70%

and 22-50 showing relatively low plasticity.

Figure 15 shows the activity of the two clays. Activity of the Tan An clay is very high with average

$A = 1.5$ and the Can Tho clay has intermediate activity about $A = 1.0$. The Can Tho clay has larger

variation of activity than the Tan An clay.

5.2 Natural water contents and liquidity indices

Natural water contents (w_n) of Tan An clay are very high ranging from 80-110%, but the liquidity indices

(LI) are less than unity with a few exception as shown in Figure 6. Can Tho clay has relatively low natural w_n 20 40 60 80 100 0 5 0 100 150 Liquidity limit w_L (%) Plasticity index I_p : Can Tho : Tan An : Hai Phong : Bangkok : Singapore : Pusan : Onsoy : Drammen : Pisa : Bothkennar : Louiseville : Ariake : Ishinomaki : Kumamoto Bay : Minamihonmoku Osaka Bay A-line : $I_p = 0.73(w_L - 20)$ B-line : $w_L - 50$ A-line B-line (CH) (CL) (MH) (ML) Vietnam Southeast Asia Japan Korea North Europe Europe(IT.,UK.) East Canada

Figure 14. Plasticity chart. $A = 1.50$ (TanAn) $A = 1.01$ (Can Tho) 0 20 40 60 80 100 0 20 40 60 80 100 Clay content under 2 μ m (%) Plasticity index I_p Tan An Can Tho

Figure 15. Activity of three clays in Vietnam.

water contents with large variation from 38% to 79% (Figure 8). Liquidity indices vary widely especially

at depths shallower than 15 m. At the depth from 7 m to 10 m where very low plastic soils are deposited,

LIs are well above unity, 1.8 as maximum and LIs below 16 m are all less than unity. It should be noted

that the data on w_n are obtained from two bore holes, one for JFP sampling and one for Shelby tube

sampling, which are only 10 m apart, showing very large fluctuation of the state of soil in the vertical

direction. Figure 16 shows initial water contents of the specimens used for various laboratory tests. The

fluctuation of water contents with depth is more prominent, even at very small depth difference. As

the liquidity index was calculated using Atterberg limits

obtained at the closest depth, this might cause

further variation of LI in the vertical variation.

Depth (m)	Physical test	UC(JFP)	CRS(JFP)	Oedometer (JFP)	UC (Shelby)	CRS(Shelby)	Oedometer (Shelby)	w _p	w _L								
0	5	10	15	20	25	30	0	10	20	30	40	50	60	70	80	90	Water content (%)

Figure 16. Initial water contents of samples for various tests: Can Tho clay.

6 ENGINEERING PROPERTIES

As reported by Man (2003), previous soil investigations on Mekong Delta clays produced strange results

on consolidation properties from the oedometer tests, such as decreasing OCR with depth to very low

value far less than unity and very large compression due to reloading to the effective vertical stress (σ'_v)

as shown in Figures 3 and 4. This could be attributed to disturbance caused by various processes of soil

sampling and testing. In order to investigate the sensitiveness of Mekong Delta clay to the sampling

disturbance, various mechanical tests were performed on samples retrieved by JFP and Shelby.

6.1 Consolidation tests

6.1.1 Compressibility

Figure 17 shows e - $\log \sigma'_v$ relations measured by constant rate of strain consolidation test (CRS) for Can

Tho clay retrieved at various depth by the JFP sampler. Except for the sample at the depth of 9.4 m,

S-shape curves with clear bent were observed, which is typical compressibility of natural clay but has

rarely been observed in the soil investigation of Vietnamese clay (Man, 2003).

The e - $\log \sigma'_v$ relations measured by CRS test for the samples retrieved by JFP sampler and Shelby

tube sampler (hereafter called JFP sample and Shelby sample

respectively) are compared both for the

Tan An clay and the Can Tho clay in Figures 18. For the Tan An clay with high plasticity, there is less

difference in the e - $\log \sigma' v$ curves for the two samples at the similar depth. Both samples at the shallower

depth show gradual decrease of void ratio with $\log \sigma' v$ and those at the deeper depth show sharp decrease

of void ratio after the consolidation yield stress (Figure 18(a)). While for the Can Tho clay with low

plasticity, the shapes of e - $\log \sigma' v$ curves are different in JFP and Shelby samples, sharp decrease after

the yield stress in the former and gradual decrease in the latter (Figure 18(b)). From Figures 18 it can be

confirmed that sampling disturbance depends not only on sampling methods but also on type of clay.

Figure 19 shows the variation of compression index C_i with consolidation pressure, that is the slope of

e - $\log \sigma' v$ curves observed in CRS tests for the JFP samples of CanTho clay. The C_i shows sharp increase to

the maximum value just after the yield stress and then gradually decreases to almost a constant value at the

stress well above the yield stress. Following the method of Tan et al. (2003), the index C_{C1} and C_{C2} were

obtained, where C_{C1} is the maximum C_i and C_{C2} is the C_i at $\sigma' v = 1,000$ kPa. These indices and their ratio

C_{C1} / C_{C2} are plotted against depth in Figure 20. At the TanAn site (Figures 20 (a) and (b)), the observed C_{C2}

are almost constant with depth, $C_{C2} = 0.66$ - 0.96 , regardless of sample types, while the compressibility

just after yield stress, C_{C1} , dramatically changes at the depth of 6 m; $C_{C1} = 0.75$ - 1.47 and $C_{C1} / C_{C2} = 1.1$ -

1.7 for the shallower depths and $C_{C1} = 2.28$ - 4.88 and $C_{C1} / C_{C2} = 2.3$ - 6.2 for the deeper depths. From the 1 z=7.4m 2
1.4 2.2 1.8 1.6 1.2 0.8 0.6 $\sigma' v$ (kPa) 1 10 100 1000 10000

Can Tho z=21.4m z=9.4m z=17.4m z=25.4m z=13.4m z=5.4m
z=11.4m v o i d r a t i o , e

Figure 17. e - $\log \sigma' v$ curves of the samples retrieved by JFP: Can Tho, CRS. 0.8 1 1 10 100 1000 1 2 3 5.61m(JFP) 5.29m(Shelby) 7.51m(JFP) 7.31m(Shelby) 3.51m(JFP) 3.26m(Shelby) 2.5 1.5 v o i d r a t i o , e (a) Tan An 0.5 1 10 100 1000 10000 $\sigma' v$ (kPa) v o i d r a t i o , e 2.2 2 1.8 1.6 1.4 1.2 0.6 $\sigma' v$ (kPa) 7.08m(Shelby) 11.63m(Shelby) 5.13m(Shelby) 7.43m(JFP) 5.43m(JFP) 11.43m(JFP) (b) Can Tho 10000

Figure 18. Comparison of e - $\log \sigma' v$ curves of samples retrieved by JFP and Shelby sampler.

difference of microstructuration at the shallower and deeper depths and the results of C 14 dating shown in

Table 1, it can be inferred that deposition period might not be proportional to depth and there should be a

gap of the period at a depth of about 6 m. From the C C1 /C C2 ratio, the difference between JFP and Shelby

samplers can be seen for the deeper samples but this difference is not clear for the shallow very young soil. At the Can Tho site, C C2 and C C1 values obtained from JFP samples are 0.39-0.53 and 0.46-3.3 as

shown in Figure 20(c). C C1 /C C2 ratio of the Can Tho clay ranges from 1.2 to 6.7 showing the tendency

of increasing with depth (Figure 20(d)). This trend becomes more clear if the data at the depth with low

plasticity index are excluded, e.g., the depth about 9.4 m. Although the results on Shelby tube samples 0 1 2 3 4 1 10 100 1000 10000 C i 3.5 2.5 1.5 0.5 z=13.4m z=11.4m z=7.4m z=5.4m z=17.4m z=21.4m z=25.4m z=9.4m Consolidation stress $\sigma' v$ (kPa)

Figure 19. Variation of compression index C i with $\sigma' v$ observed by CRS: JFP samples, Can Tho clay. 0 5 10 15 20 25 30 0 1 2 23 4 0 4 6 8 2 3 4 5 6 7 8 0 2 3 4 5 1 2 4 6 C C1 , C C2 C C1 /C C2 0 Cc1(JFP) Cc1(Shelby) Cc2(JFP) Cc2(Shelby) JFP Shelby D e p t h (m) (a) C C1 and C C2 for Tan An clay (b) C C1 /C C2 for Tan An clay C C1 /C C2 C C1 , C C2 Cc1(JFP) Cc1(Shelby) Cc2(JFP) Cc2(Shelby) Shelby JFP D e p t h (m) (c) C C1 and C C2 for Can Tho clay (d) C C1 /C C2 for Can Tho clay

Figure 20. Variation of C_c , C_u , C_c / C_u with depth for JFP and Shelby samples: CRS. 0 1 2 OCR JFP 0 5 10 15 20 25 30 0 100 200 2 3 4 5 6 7 8 50 JFP σ'_v σ'_v 2 OCR JFP p_y (kPa) D e p t h (m) D e p t h (m) Shelby 1 Shelby (a) p_y of Tan An clay (b) OCR of Tan An clay (c) p_y of Can Tho clay (d) OCR of Can Tho clay p_y (kPa) 1000 0 JFP Shelby Shelby

Figure 21. Variation of p_y and OCR obtained from CRS tests for Tan An and Can Tho clays.

are limited in the shallower depth, the distinct difference between JFP and Shelby tube samples can be

confirmed from the ratio of C_c / C_u . The largest C_c / C_u ratios observed for the Tan An and Can Tho clays, more than 6, are even larger

than the ratio of Pleistocene marine clay at the New Kansai Airport site, which is well known clay with

high degree of structures (Tanaka et al., 2003), suggesting the very high structuration of the two clays,

despite very recent deposition about 6,000 BP as shown in Table 1.

6.1.2 Consolidation yield stresses and OCRs

Consolidation yield stress (p_y) and overconsolidation ratio (OCR) obtained from the CRS tests are shown

in Figures 21. The OCR of Tan An clay (Figure 21(b)) changes at the depth of 6 m, lower OCR (1.1-1.6)

at the shallower depth and higher OCR (1.8-2.2) at the deeper depth. The difference of OCR between

the two samples is not noticeable for the Tan An clay. For the JFP samples, the p_y of Can Tho clay increases almost linearly with depth, resulting in relatively

constant OCR with depth ranging from 1.5 to 1.8, except at $z = 9.4$ m (Figures 21 (c) and (d)). While

for the Shelby tube sample, there is no clear trend of p_y and the OCRs are similar to those of the JFP

samples at some depths, but much smaller than JFP at some

other depths, even less than 1. From the

above comparisons, it can be said that very small OCRs reported in the previous soil investigations (see

Figure 3) could be partly attributed to the poor sampling quality.

6.1.3 Intrinsic compression behavior

For some depths, incremental loading oedometer tests were conducted for the reconstituted sample.

The specimen was prepared from the slurry with water content of 1.5 times the liquid limit. For the

Tan An clay, the slurry was first consolidated in the preconsolidation mold with $\sigma'_{v0} = 50$ kPa and then

the preconsolidated clay was inserted into the oedometer ring. While for the Can Tho clay, as the

preconsolidated clay block is too soft to stand without confinement, the slurry was preconsolidated in 0.5 1 1.5 2 2.5 3 1 10 100 1000 10000 σ'_v (kPa)

V o

i d

r a

t i o

, e 7.5m 3.5m 7.5m 3.5m Reconstituted Intact JFP S. (CRS)
0.4 0.6 0.8 1 1.2 1.4 1.6 1.8 2 2.2 1 10 100 1000 21.1m
17.1m 7.3m 5.1m 11.1m 21.4m 17.3m 7.4m 5.4m 11.4m σ'_v
(kPa) V o i d r a t i o , e Reconstituted (a) Tan An Intact
JFP S. (CRS) 10000 (b) Can Tho

Figure 22. Intact and intrinsic compression curves of Tan An and Can Tho clays: JFP samples.

the oedometer mold with $\sigma'_{v0} = 20$ kPa. The e-log σ'_v curves of reconstituted samples are compared to

those of JFP samples in Figures 22. For the Tan An clay, the e-log σ'_v curves of reconstituted and JFP

samples tend to merge at higher stress. While for the Can

Tho clay, in general there are large differences

between reconstituted and JFP samples, no clear tendency of convergence of the curves at the higher

stress. The difference of void ratio at the higher stress for the reconstituted samples is larger than the

JFP samples, especially for the samples at depths shallower than 11.5 m. It can be also confirmed from

Figure 22 (b) that the intrinsic properties of Can Tho clay changes with depth significantly.

In-situ void index (I_{v0}) was evaluated using the normally consolidated line in the e -log σ'_v curves of

reconstituted samples as the intrinsic compression curves, following the definition proposed by Burland

(1990). The I_{v0} is plotted to in-situ effective vertical stress with intrinsic compression line (ILI) and

sedimentary compression line (SCL) proposed by Burland (1990) in Figure 23. The I_{v0} of Tan An clay

at $z = 3.5$ m is in the middle position between ICL and SCL and that at $z = 7.5$ m is on or slightly below

SCL. This difference of I_{v0} gives large difference of structuration as shown in Figures 18, 20 and 21.

The I_{v0} indices of Can Tho clay at the shallower depths are well above SCL and those at the deeper

depths are on or slightly over SCL as shown in Figure 23. However, degrees of structuration in terms of

C_c / C_c and OCR are increasing with depth and less depth dependent respectively (see Figures 20(b)

and 21(b)). Very large I_{v0} values might be attributed to the heterogeneity in the vertical direction as

shown in Figures 12. The reconstituted clay was prepared from a 50 mm height sample. If the coarse and

fine grain materials from different thin layers are mixed, the mixture becomes a well graded material of

which intrinsic properties is different from the natural one with lower void ratio.

6.1.4 Coefficient of consolidation

Figure 24 shows the variation of coefficient of consolidation (c_v) with consolidation stress σ'_v obtained

from CRS tests on JFP samples of Can Tho clay. Reliable c_v could not be obtained from Tan An clay,

which is probably because of the randomly distributed holes in the sample (Figure 10). The c_v value

drastically drops at the stress near p_y and then increase after the minimum value. Figure 25(a) shows the

depth variations of averaged c_v values from the in-situ vertical stress σ'_v to the p_y , which is denoted by

c_{v1} , and c_v values at $\sigma'_v = 1000$ kPa, denoted by c_{v1000} . The ratios of c_{v1}/c_{v1000} are shown in Figure 25(b).

For the JFP samples, c_{v1} is very much larger than c_{v1000} , except for the samples at $z = 9.4$ m of which c_{v1}

is smaller than c_{v1000} . While for the Shelby tube samples, there is no consistency of relative magnitude

between c_{v1} and c_{v1000} or the ratio c_{v1}/c_{v1000} indicating the large variation of sample quality. -1 0 1 2
3 4 5 6 10 I v 0 1000 Tan An clay z=3.5m Shallow Can Tho
clay z=5 -11m Deep Can Tho clay z=17 -21m I C L (B u r l a
n d , 1 9 9 0) S C L (B u r l a n d , 1 9 9 0) Tan An
clay z=7.5m σ'_v (kPa) 100

Figure 23. Relationship between void index I_{v0} and σ'_v for Tan An and Can Tho clays. 0.001 0.01 0.1 1 10 100
1000 10000 Effective vertical stress σ'_v (kPa) c_v (m² /
d a y) z=5.4m z=7.4m z=9.4m z=11.4m z=21.4m z=13.4m
z=17.4m z=25.4m

Figure 24. Variation of c_v with σ'_v observed by CRS: JFP samples, Can Tho clay.

6.1.5 Change of void ratio by recompression to the in-situ vertical stress, e_e

Change of void ratio or volumetric strain caused by the recompression to in-situ vertical stress, σ'_{v0} , can

be used as indicators of sample quality (Andresen and Kolstad, 1979; Lacasse and Berre, 1988; Lunne

et al, 1997). The change of void ratio normalized by the initial void ratio, $\Delta e/e_0$, are plotted to the

depth of the sample in Figure 26. For the Tan An site, the $\Delta e/e_0$ values range from 0.033 to 0.053 for

the samples shallower than 6 m, except for the Shelby tube sample at $z = 4.3$ m, while for the samples

deeper than 6 m, the values range from 0.13 to 0.35. Neglecting the exception at $z = 4.3$ m, it can be

reconfirmed that the sample quality difference between the two types of sampler are not significant for

the very soft, less structured clay at the shallower depth compared to that for the relatively stiff clay with

structuration at the deeper depth as discussed for the C C1 /C C2 in Figure 20(b). For the Can Tho site, the $\Delta e/e_0$ values of JFP samples are from 0.017 to 0.037 within the range of

very good or excellent quality, defined by Lunne et al. (1997) except for the samples at $z = 9.4$ m with $\Delta e/e_0$ 0.5 10 15 20 25 30 1 cv JFP Shelby c v1, c v1000 (m²/day) 0.001 0.01 0.1

d e

p t

h

(m

) v1(JFP) cv1 (Shelby) cv1000(JFP) cv1000(Shelby) c v1 / c v1000 0.1 10 h lby (a) c v1 and c v1000 (b) c v1 / c v1000 1 100

Figure 25. Variation of c_{v1} , c_{v1000} and c_{v1}/c_{v1000} with depth for JFP and Shelby samples: CRS, Can Tho clay. 0 5 10 15 20 25 30 2 3 4 5 6 7 8 (1) (2) (3) (4) (b) Can Tho (a d e p t h (m) JFP Shelby 0 0.04 0.08 0.12 0.16 0.2

0.04 0.08 0 0.12 0.16 0.20 $\Delta e/e_0$ $\Delta e/e_0$ d e p t h (m) Tho
)Tan An Sample quality (Lunne,1997) (1)Very good or
 excellent (2)Good to fair (3)Poor (4)Very poor

Figure 26. Sample quality in terms of compression due to
 reloading to σ'_{v0} : CRS.

e/e_0 values of about 0.6 (Figure 26(b)). Comparing Figure
 26 with the figures showing the sample

quality of natural soils, Figures 20 (b) and (d) on C_{c1}/C_{c2} , Figures 21(b) and (d) on OCR, and Figure

25(b) on c_{v1}/c_{v1000} , it can be confirmed that the $\Delta e/e_0$
 0 value is a good indicator of the sample quality

and the samples with the $\Delta e/e_0$ values over 0.07,
 corresponding to poor quality (Lunne et al, 1997),

might yield very low OCR even less than 1 and relatively
 low C_{c1}/C_{c2} values. It should be noted that

the samples could not be retrieved using the Shelby tube
 sampler at the depth of lowest plasticity index,

about 8-10 m.

6.2 Unconfined compression tests

Axial stress-strain curves obtained by unconfined
 compression (UC) tests are shown in Figure 27.

Unconfined compression strength, q_u , residual suction
 normalized by in-situ vertical stress (σ'_{v0}) and

normalized undrained strength, $(q_u/2)/\sigma'_{v0}$ measured by
 UC tests are summarized in Figure 28. The suction

of the test specimen was measured by using the way
 described by Tanaka et al. (2002). The q_u values at the

Figure 27. Comparison of axial stress and strain curves of
 UC tests for JFP and Shelby samplers.

two sites increase almost linearly with depth. At the Tan An
 site, the normalized strength, $(q_u/2)/\sigma'_{v0}$, tends

to increase with depth from about 0.3 at $z = 3.5$ m to 0.6
 at $z = 7.5$ m, while at the Can Tho sites no clear

trend of the normalized strength with depth but showed a large variation from 0.16 to 0.52. The difference

in these parameters between the JFP and Shelby samples are not distinguished clearly. Figures 29 show the

relationships between the $(q_u/2)/\sigma'_{v0}$ and the normalized suction. Although the suction measurement for

the Tan An clay might not be so reliable because of the holes in the sample, a trend of increasing $(q_u/2)/\sigma'_{v0}$

with the normalized suction can be seen in Figure 29(a). The more apparent trend can be seen between the

q_u and the residual suction until the normalized suction becomes about 0.2 for the Can Tho clay. But the

normalized strengths are almost constant about 0.4 for the normalized suction over 0.2. This indicates the

residual suction or effective stress of the sample as a quality indicator for the undrained shear strength. Figure 30 show the failure strains, ϵ_f , and the secant moduli divided by the undrained shear strength,

$E_{50}/(q_u/2)$. Because E_{50} depends on strength and consolidation pressures, it is normalized by these

parameters and can be used as an indicator of sample quality (e.g., Nakase et al, 1985). From Figures 30

it is also confirmed that at the Tan An site there is less difference between these parameters of the two

samples at the shallower depth and clear difference at the deeper depth. The difference becomes much

more apparent for the Can Tho site.

6.3 K_0 consolidation tests for Can Tho clay

For the JFP samples of the Can Tho clays, K_0 -consolidation tests were conducted using the triaxial test

apparatus (Watabe et al., 2003). The specimen with the dimension of 35 mm in diameter and 85 mm

in height was first consolidated with isotropic stress of $\sigma'_{v0}/3$. After this preliminary consolidation, K_0

consolidation was performed maintaining zero horizontal strain till the vertical stress reached three

times of σ'_{v0} . Variation of the stress ratio σ'_h / σ'_v with the normalized vertical stress σ'_v / σ'_{v0} in the K_0

consolidation test are shown in Figure 31. At all depths of the samples tested, the stress ratios first

decrease with increasing vertical stress and converge to constant values, which can be regarded as K_0

values of normally consolidated conditions, K_{0NC} . The values of K_{0NC} ranged from 0.44 to 0.59, similar

to those of normal marine clay deposits in Japan (Watabe et al, 2003). According to Watabe et al.

(2003), the structuration of the clay can be distinguished from the variation of the stress ratio with the

normalized vertical stress. The stress ratio of the clay with high structuration, such as aged Pleistocene suction/ σ'_{v0} 0 0.2 0.4 0.6 0.8 JFP Shelby q_u (kPa) 2 3 4 5 6 7 8 0 20 40 60 80 d e p t h (m) suction/ σ'_{v0} q_u (kPa) ($q_u/2$)/ σ'_{v0} 0.20 0.4 0.6 0.8 0.0 0.2 0.4 0.6 JFP Shelby ($q_u/2$)/ σ'_{v0} 0.0 0.2 0.4 0.6 0 5 10 15 20 25 30 0 5 0 100 150 d e p t h (m) (a) Tan An (b) Can Tho

Figure 28. Results of UC tests for JFP and Shelby samplers. 0.0 0.1 0.2 0.3 0.4 0.5 0.6 0.7 0.2 0.6 0.8 0 0.1 0.2 0.3 0.4 0.6 0 0.6 0.8 0.0 0.4 suction/ σ'_{v0} ($q_u/2$) / σ'_{v0} o JFP(z < 14m) JFP(z > 14m suction/ σ'_{v0} 0.5 0.7 0.2 0.4 ($q_u/2$) / σ'_{v0} JFP Shelby (a) Tan An (b) Can Tho Shelby

Figure 29. Relationship between suction and undrained strength of UC tests.

clay, decreases the minimum values at the vertical stress slightly above the in-situ stress (σ'_{v0}) and then

turn to increases to K_{0NC} . While for the clay with less structuration, such as young normally consolidated

clay, sandy soils, and reconstituted clay, the ratio monotonically decreases to K_{0NC} . Comparing Figure 31

with Figure 20(d) the above features can be confirmed for the Can Tho clay. The sample at $z = 9.3$ m

with the minimum C_{c1}/C_{c2} ratio shows no minimum values in the variation. On the other hand, the

samples at deeper depth with large C_{c1}/C_{c2} ratio show the minimum value and marked increase of the

stress ratio.

K₀ consolidated undrained compression and extension tests (CK₀ UC and CK₀ UE) were also conducted

for the specimen anisotropically consolidated with the stress ratio $\sigma'_h/\sigma'_v = K_{0NC}$ and $\sigma'_v = \sigma'_{v0}$. Stress path

and deviator stress axial strain curves obtained from CK₀ UC and CK₀ UE tests are depicted in Figure 32.

Marked peaks of deviator stress were observed for the samples at deeper depths, but not for the sample at JFP Shelby 0 5 10 15 20 25 30 0 100 200 300 400 2 3 4 5 6 7 8 10 15 (a) Tan An (b) Can Tho JFP Shelby E 50 / (q u / 2) 0 5 0 100 200 300 400 10 150 5 ϵ_f (%) ϵ_f (%) E 50 / (q u / 2)

d e

p t

h

(m

) d e p t h (m) JFP Shelby

Figure 30. Sample quality in terms of ϵ_f and E_{50}/c_u from UC tests. 0 0.2 0.4 0.6 0.8 1 0 1 2 3 σ'_v/σ'_{v0} σ'_h/σ'_v $z = 5.3m$ $z = 7.3m$ $z = 9.3m$ $z = 11.3m$ $z = 13.3m$ $z = 15.2m$ $z = 17.2m$ $z = 21.2m$

Figure 31. Relationship between σ'_h/σ'_v and σ'_v/σ'_{v0} observed in K₀ consolidation in triaxial test for Can Tho clay:

JFP samples.

shallower depths especially $z = 9.6$ m. Strength anisotropy s_{u_CKoUE}/s_{u_CKoUC} are about 0.8 at the shallower

depths ($z < 10$ m) and 0.6 at the deeper depth ($z > 11$ m).

6.4 Undrained strengths and sensitivity

For the Can Tho clay, field vane tests (FV) and laboratory vane tests (LV) were conducted to evaluate

the undrained strength and sensitivity. The size of vane blade of FV is 80 mm in height and 40 mm in

width and that of LV is 30 mm in height and 15 mm in width. Rotational rate of the vane tests were 6

degree/min for both FV and LV tests. The laboratory vane blade was inserted to the sample in the sampler

to the depth of 85 mm. In Figure 33, the undrained shear strengths obtained from the FV and LV tests

are plotted with the depth as well as the strength estimated from the other tests, UC tests, CK $\emptyset$ UC and

CK $\emptyset$ UE tests, and constant volume direct shear test (DST). No correction was done for the undrained

strength from the vane test according to plasticity index, such as the way proposed by Bjerrum (1972),

because the plasticity indices of Can Tho clay are relatively low. The undrained shear strength from

the FV and LV tests agreed well except at the depth of 9.3 m and 10.3 m where low plastic soils with

thin sand seams deposit. For the shallower depth less than 12 m, the vane strength agreed well with the

strength from DST and higher than $q_u / 2$ from the JFP samples. It is interesting that the field vane shear $\emptyset$ 6 93 12 15 21.5m 17.5m 11.6m 9.6m 5.6m 5.6m 9.6m 11.6m 17.5m 21.5m Axial strain ϵ_a (%) -100 -150 -50 0 50 100 0 50 100 150 K $\emptyset$ line 21.5m 11.6m 9.6m 5.6m 21.5m 11.6m 9.6m 5.6m 150 -100 -150 -50 0 50 100 150 Mean principal effective stress p' (kPa) D e v i a t o r s t r e s s q (k P a) D e v i a t o r s t r e s s q (k P a) 17.5m 17.5m

Figure 32. Results of triaxial CK $\emptyset$ U test for Can Tho clay: JFP samples. 0.0 5.0 15.0 20.0 25.0 30.0 0 20 40 60 80 10.0 D e p t h (m) Lab_V undisturbed (JFP) Lab_V

disturbed (JFP) Lab_V undisturbed (Shelby) Lab_V disturbed (Shelby) Filed_V Field_V disturbed $q_u/2$ (JFP) CK0UC CK0UE DST Undrained shear strength (kpa)

Figure 33. Undrained shear strength obtained from various tests for the Can Tho clay.

strength at the depth about 9.5 m is the largest among those obtained from various tests. For the deeper

depth, however, the vane strengths are smaller than that from DST and almost equivalent to the $q_u/2$.

Sensitivity obtained from the field vane and laboratory vane tests are plotted with the depth in

Figure 34. Depth variation of liquidity index, LI, is also shown in the figure. Sensitivity variation

with depth at the Can Tho site shows similar trend of LI variation with depth. However, despite the large

liquidity index greater than 1.0 and high degree of microstructuration discussed above, the sensitivity

of Can Tho clay derived from the vane tests are relatively small ranging from 2.5 to 4.0. 0 5 10 15 20 25 30 2.0 2.5 3.0 4.0 4.5 0.4 1.6 LI 1.0 3.5 Sensitivity d e p t h (m) Lab_V (JFP) Lab_V(shelby) Field_V LI

Figure 34. Depth Variation of sensitivity obtained from field and laboratory vane tests and liquidity index. 0 5 10 15 20 25 30 0 20 40 60 80 Undrained shear strength(kPa) d e p t h (m) $q_u/2$ (JFP) CPT(Nkt = 10) CPT(Nkt = 14) FVT CK0UC CK0UE DST

Figure 35. Comparisons of undrained shear strength obtained from CPT with those by the other tests.

6.5 Field tests

6.5.1 Piezocone cone penetration tests

For the soil profiles with large vertical heterogeneity like the Can Tho site, the cone penetration test

has a strong advantage. The continuous measurement of quantities, such as the tip resistance and the

pore water pressure behind the cone can capture the entire

variation of soil profiles, some of which

might be missed in the procedure of the sampling and laboratory testing of a certain intervals. However,

interpretation of the cone test results is not straightforward. The undrained strength profiles estimated

from the piezocone tests are given in Figure 35 with the strengths obtained from the other tests. The

cone factor $N_{KT} = 10$ and 14 are assumed for evaluating the undrained shear strength. The field vane

strength (s_{uFV}) at the shallower depth agrees well with the undrained strength from the CPT (s_{uCPT}) with $N_{KT} = 10$ but at the deeper depth the s_{uFV} becomes closer to the s_{uCPT} with $N_{KT} = 14$. The undrained shear strength of constant volume direct shear test (s_{uDST}), which is close to the average of undrained strength of CK $\emptyset$ UC and CK $\emptyset$ UE tests, has the opposite trend to the s_{uFV} in the relation to the s_{uCPT} . The possible reason of the difference is the effect of partial drainage which might take place during the vane tests at the shallower portion with low plastic soils and thin sand seams. The strength estimation of the low plastic soils, which might be categorized as intermediate soils, is a difficult issue, including the question if the undrained strength is a proper parameter in stability analyses. Application of cone test

Figure 36. Results of dilatometer tests at Tan An and Can Tho sites.

$N_{KT} = 10$ but at the deeper depth the s_{uFV} becomes closer to the s_{uCPT} with $N_{KT} = 14$. The undrained

shear strength of constant volume direct shear test (s_{uDST}), which is close to the average of undrained

strength of CK $\emptyset$ UC and CK $\emptyset$ UE tests, has the opposite trend to the s_{uFV} in the relation to the s_{uCPT} . The

possible reason of the difference is the effect of partial drainage which might take place during the vane

tests at the shallower portion with low plastic soils and thin sand seams. The strength estimation of the

low plastic soils, which might be categorized as intermediate soils, is a difficult issue, including the

question if the undrained strength is a proper parameter in stability analyses. Application of cone test

on this type of soils needs further studies, which is very crucial for the Mekong Delta clay with rapid

deposition including coarser materials.

From Figure 35, it can be said that very good quality sample, like the ones retrieved at the deeper

depths, is crucial for UC tests to evaluate a reasonable cone factor.

6.5.2 Flat dilatometer tests

Results of Marchetti's flat dilatometer test (DMT) done at Tan An site and Can Tho site are summarized

in Figure 36. Overall trend of engineering properties at the two sites can be captured well by the DMT,

but very thin sand layer or seams at the depth of about 15 m at the Can Tho sites could not be detected

by the DMT, although it indicates small increases of material index I_D and dilatometer modulus M_D . K_0

values calculated using equation (1), which is given by Marchetti (1980), are also shown in Figures 36.

The K_0 DMT values of the Tan An Clay are in the reasonable range from 0.45 to 0.7 for the depth of very

soft high plastic and from the depth below 8 m, the values jump up to nearly 2.0. Due to the high stiffness,

sampling could not be done at the depth below 8 m, however, it is inferred from the DMT results and

the CPT results shown in Figure 7 that the soil in the stiff layer is hard slit or clay with very large OCR.

There is a gap in the depth variation of K_0 DMT of Can Tho clay at depth about 11.5 m. K_0 NC obtained

from the K_0 consolidation tests (Figure 31) and K_0 OC calculated by the equation (2) proposed by Mayne

and Kulhawy (1982) are shown in Figure 36(b) with the K_0 DMT values of Can Tho clay.

Although the gap of K_0 DMT cannot be seen in the results of the triaxial K_0 consolidation tests, the K_0

values obtained from the two methods agree quite well. The K_0 DMT values at the depths about 10 m are

unrealistically small less than 0.3. For evaluating the K_0 value by using field tests, further studies are

needed especially for the low plastic soils.

7 CONCLUSIONS

This paper has presented soil characterization studies done for the alluvial deposits at the Tan An and Can

Tho sites in Mekong River Delta. The Tan An clay, which has been deposited at the riverside of meandering

Bam Co Tay River, is highly plastic and very soft with relatively uniform physical properties to a depth of

about 8 m. The Can Tho soft clay has been deposited at the flooding area beside the levee of Hau River, one

of the major streams of Mekong River. It has significant heterogeneity in the depositional direction, hav

ing relatively low plasticity with thin sand seams, which might be created due to large floods. Depositional

speed of the Can Tho clay is much faster than that of the Tan An clay. C 14 dating of the Tan An clay at depth

of 6.5 m and the Can Tho clay at depth of 19.5 m are almost the same, 6,300 BP. Although mechanical

and physical properties of the two clays are quite different, both clays have developed high degree of

microstructures, which has been confirmed from CRS tests. The ratios of maximum compression index

just after the consolidation yield stress to that at the very large consolidation pressure are more than 6 for

both clays if excellent quality of sample can be retrieved. For the non uniform and very thick soft deposit

like the Can Tho clay, continuous soil sounding using

piezocone is essential for characterizing the site. Wide varieties of soft soils with different physical and chemical properties prevail in the huge area

of Mekong River Delta. Continuous soil investigation studies and practices with good quality sampling,

such as thin-walled tube sampler with fixed piston, and laboratory testing as well as appropriate field

testing are highly recommended, which can greatly contribute to the proper development in Mekong

River Delta.

ACKNOWLEDGEMENTS

The authors would like to acknowledge collaborators in the soil characterization studies, Dr. P. H. Giao

(AIT), Mr. B. T. Man and Dr. N. H. Quan (HCMCUT), Mr. T. H. Hung (USCO), Mr. C. N. Sung and

Mr. D. C. Thuan (TEDI), Mr. K. Ohshiro and Mr. Y. Yuasa (Tokyo Tech), Mr. Y. Shiraishi (Nippon Koei

Co., Ltd.), Dr. T. Fukasawa and Mr. H. Saegusa (Toa Corp.), and Mr. T. Ueda (Saeki Kensetsu Kogyo

Co., Ltd.). The authors also are grateful to Taisei-Kajima-Nippon Steel Joint Operation for rendering the

various supports during the field investigation at Can Tho bridge construction site. The authors gratefully

acknowledge the funding for this research from Japan Society for the Promotion of Science (JSPS) as

Grant-in-Aid program.

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Geotechnical properties of Hachirogata Clay

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ABSTRACT: A series of site investigations was carried out

at the site of Hachirogata, where Holocene

Clay is thickly deposited. The distinguishing feature of Hachirogata Clay is high index properties; for

example, its plasticity index exceeds 100. The reason for such large index properties may be attributed

to the presence of clay minerals with high activity and large organic content. In addition, another

possible reason is the presence of a large proportion of macrofossils, especially diatoms, which have

been observed by Scanning Electron Microscope (SEM). The representative size of these diatoms is

about 10 μm , which is classified into silt size. However, diatoms have a large capacity for holding

water, because of their porous structure. This paper deals with interesting properties of Hachirogata

Clay, compared with other Japanese soils and overseas soils based on the data accumulated by the author

through his site investigations.

1 INTRODUCTION

The Hachirogata site is located in Akita prefecture, in the northern part of Honshu Island, Japan. Hachi

rogata was the second largest lake in Japan before its reclamation in the 1970's as cultivated land for

a rice paddy. The investigated site is located at the center of the reclaimed lake, where the soft clay is

thickly deposited to more than 45 m. PARI (Port and Airport Research Institute, formally PHRI, Port

and Harbor Research Institute) carried out a series of site investigations to study geotechnical properties

of the soft clay, especially sampling quality retrieved by several types of samplers, such as the Japanese

standard sampler (fixed piston thin wall sampler), Shelby tube sampler and Laval sampler (Oka et al.

1996).

The distinguishing feature in the geotechnical properties of Hachirogata Clay, compared with those

of soft clays at other sites, is its high index properties; for example, its plasticity index (I_p) exceeds

100. The author has not previously experienced a clay with I_p more than 100, although I_p for Japanese

marine clays is generally large, compared with that of other overseas clays. Even in the literature, studies

on clay with such high I_p can be rarely found except for Mexico clay, which is well known for large

settlements in the foundation of high-rise buildings in Mexico City (see, for example, Mesri et al. 1975;

Dias-Rodriguez 2003).

It is found from observation by Scanning Electron Microscope (SEM) that Hachirogata Clay contains

a large proportion of microfossils, especially diatoms. It is anticipated that if soil contains diatoms,

its index properties as well as its natural water content can become very large, because of large pores

within diatom particles (Tanaka & Locat 1999; Locat & Tanaka 2001; Shiwakoti et al. 2002). This paper

presents geotechnical properties of Hachirogata Clay with high I_p and compares them with those having

moderate I_p .

2 GEOLOGICAL PROCESS OF HACHIROGATA LAKE

Hachirogata is located about 30 km north of Akita city, facing the Japan Sea, as shown in Fig. 1. The Oga

peninsula, which is located in the west side of the site, used to be an isolated island created by volcanic

activities in the late Pleistocene era. This island was connected by sand banks in the west and south

sides to the main land, prior to the Holocene era. The area surrounded by the sandbanks and the Oga

island became the lake called "Hachiro-gata" ("gata" means lagoon in Japanese), becoming the second

Figure 1. Location of site investigation for Hachirogata Clay.

largest lake, after Biwa Lake in Japan, before the lake was finally reclaimed. The clay layer investigated

in this report was deposited under blackish or lacustrine environmental conditions, since sea water was

coming through only a restricted narrow channel in the south sandbank. The lake prior to reclamation

was famous for plenty of brackish fish. The water depth of the Hachirogata Lake was relatively shallow

and ranged from 4 to 5 m. The work of reclamation was started in 1957 by constructing dikes, whose

length is 52 km in total along the shore of the original Hachirogata Lake. It took as long as 20 years for

reclamation and the work was completed in 1977 (more information on the geological process may be

referred to Shiraishi 1990). The thickness of clay layer varies with locations, being deepest at the center and gradually becoming

thinner towards the fringe of the lake. The investigated site is located nearly at the center point, where

the thickness of the soft clay is as much as 45 m.

3 MAIN PHYSICAL AND CHEMICAL PROPERTIES

Figure 2 shows typical physical and chemical properties of the Hachirogata Clay. The soil samples were

recovered from as deep as 45 m. Below a depth of 45 m, a stiff sand layer was found. A soil profile

measured by Piezocone Penetration Test (CPTU) is shown in Fig. 3, where q_t , u and f_s are the point

resistance considering the effective area ratio of the cone, the pore water pressure and the skin friction,

respectively. As shown in the profile obtained by CPTU, the layer is very homogeneous and any high permeable

layers are not found. The original ground surface is covered by sandy soil to 3 m depth. As shown in

grain composition, the soil largely consists of silt and clay particles (where the boundary particle size is

2 μm). The grain composition of the upper layer down to 8 m depth is mainly silt, then the clay content

gradually increases with depth and the clay content is the largest at about 30 m depth. On the other hand,

index properties such as liquid limit (w_L) and plastic limit (w_P) decrease with depth in spite of an increase

in the clay content. The natural water content (w_n) is nearly equal to w_L down to a depth of 26 m, in

another word, the liquidity Index (I_L) is 1.0. However, w_n becomes smaller than w_L below a depth of

30 m, where w_L again increases with depth. Deposition age for the sediments was measured by isotope of ^{14}C and it was found that the depositional

age at 5, 19 and 35 m depths are 3500, 6300 and 8000 year Before Physics (BP), respectively. These

dates indicate that the investigated soil layer in this study was deposited in the Holocene era, i.e., after

the ice age. Considering these deposited ages and depths of the measured age, it is estimated that the

rate of deposition gradually decreased with time: 10 mm/year (8000 BP-6300 BP); 5 mm/year (6300

BP-3500 BP); 1.6 mm/year (3500 BP-the present). The speed for depositing these layers is considerably

faster compared with Holocene deposits in other areas in Japan. Salinity of liquid in the pores of the soil is shown in the right end of the column of Fig. 2. The salinity,

being presented by content of chlorite to the dry weight of soil (CH), slightly varies with depth and the

Depth (m)	CH (%)	W _n (%)	W _L (%)	P (%)	Water Content (%)	Ignition Loss (%)
0	20	40	60	80	100	40
30	20	10	0	0	50	100
150	200	250	300	0	5	10
15	0.00	0.05	0.10	0.15	Sand	Clay (<2μm)
					Silt	6,300BP
						8,000BP
						3,500BP

D

e p

t h

(m

) Content (%) w_n w_L w_P Water Content(%) Ignition Loss
 L i Organic Carbon Content OC Content of Chloride 0 5 10
 15 CH (%) (%) (%)

Figure 2. Physical and chemical properties at the site of Hachirogata.

Depth (m)	q _t (kPa)	u (kPa)	f _s (kPa)
0	500	1000	1500
2000	40	30	20
10	0	0	200
400	600	800	1000
-5	0	5	10
15	20	25	30

Figure 3. Soil profile measured by Cone Penetration Test.

largest CH is measured at the middle point of the layer. This depth corresponds quite well with the depth

where the I L becomes suddenly smaller (w_n becomes smaller than w_L). It is inferred that at this time, the

depositional environment may have changed probably due to creation of the sandbank which separated

the lake from the sea. In general, the relatively small CH compared with sea water indicates that the

environmental deposition in the soil layer may have been brackish, which is consistent with the diatom

species observed by Scanning Electronic Microscope (SEM), as will be discussed later.

Index properties for Hachirogata Clay are extremely large through all depths, especially near the

ground surface. The activity for various soils investigated by the author's group is indicated in Fig. 4. In

this figure, clay contents (the diameter of soil particles is less than 2 μm) are plotted on the horizontal

axis and I_p on the vertical axis, so that the slope in the two values presents the activity. Although it is well

known that the activity (A_c) for Japanese clays is, in general, much larger than that for other overseas
 0 10 20 30 40 50 60 70 20 40 60 80 100 120 140 160 P l a s t i c i t y I n d e x , I_p Clay Content, <2 μm (%) Hachirogata Singapore Drammen Bothkennar Ariake Luiseville Bangkok Pusan Yamashita

Figure 4. Activity of various soils in the world.
 0 2 4 6 8 10 12 14 16 0 2 4 6 8 10 12 OC O r g a n i c C a r b o n C o n t e n t O C (%) Ignition Loss, L_i (%) =1.2(L_i 5)

Figure 5. Relation between L_i and OC.

clays, the activity for Hachirogata Clay is extremely large, especially at shallow depths (A_c is more than

5). When depths are deeper than 15 m, their activity becomes close to that for other Japanese clays (in

the figure, Ariake and Yamashita are Japanese clays). The organic matter content was investigated by two types of tests: ignition loss (L_i) and organic carbon

content (OC), following the Japanese Industrial Standard (JIS) or the Japanese Geotechnical Society

(JGS) Standards (JIS A1226:2000 for L_i and JGS0231-2000 for OC). There exists a clear correlation between L_i and OC, as shown in Fig. 5. When L_i exceeds 5%, OC

increases with the increase in L_i . This relation suggests that 5% of L_i is caused by loss not attributed to

the organic matter, i.e., dehydration of clay minerals in the form of absorbed water or water of hydration
 6 8 10 12 14 40 60 80 100 120 140 160 P l a s t i c i t y I n d e x , I_p Ignition Loss, L_i (%)

Figure 6. Relation between I_p and L_i

due to high temperature of 750 °C used in the test to determine OC. Also I_p can be correlated with L_i as

shown in Fig. 6. Large index properties for Hachirogata

Clay may be attributed to the large content of

organic matter. This point will be discussed later in more detail.

Figure 7 shows X-ray diffraction charts for Hachirogata Clays at four different depths. The tested

grain size of all specimens was less than 2 μm . The specimens were prepared by three different treatment

methods: natural (denoted by N in the figure); Glycolation (G) treated; heated at 550 ° C (H). It can be

seen that the Hachirogata Clay might contain a lot of smectite, whose activity is very high. Locat et al.

(1996) have also shown that main clay mineral in Japanese clays is smectite. The large index properties

and high activity of Hachirogata Clay may be attributed to the presence of a large amount of smectite. It

is reported, however, that the ϕ' values for pure smectite are smaller compared to that of illite or kaolin

whose I_p are relatively small (see for example, Mitchell 1976). However, as described later in more

detail, ϕ' of Hachirogata Clay is large, compared with other soils having low I_p .

4 INFLUENCE OF MICROFOSSILES, ESPECIALLY DIATOMS

Figure 8 shows the microfabric of Hachirogata Clay at a depth of 6 m, observed by Scanning Electronic

Microscope (SEM). Large numbers of diatom skeletons with the shape of a hollow cylinder can be seen

in the figure: they are diatoms. It is anticipated that if a soil contains a lot of diatoms, the large sized

pores in their skeletons lead to high water content. It is seen from the figure that the size of the diatoms

is almost 10 μm , being the size of silt. It has been believed that due to electric charges on the surface of

clay particles, a volume of water is attracted by the clay

particles, and this amount of water is different

for different clay minerals. If soil contains diatoms, however, a large part of the water held is entrapped

in the pores of their skeletons, which may be classified as silt by the grading test. Therefore, it may be

inferred that the Atterberg limits are strongly influenced by the presence of diatoms. In addition, diatoms

may affect mechanical properties such as ϕ' value, because of their rough and angular shape as shown

in Fig. 8.

The influence of diatoms on the physical, hydraulic as well as mechanical properties have been studied

by artificially mixing ordinary soil with diatomite soil by, for example, Tanaka (2002); Shiwakoti et al.

(2002). It was found from these studies that with the increase in content of diatoms, both w_L and I_p

increase even though the clay content decreases. These behaviors are completely different from those

of sandy or silty soils in which, with the decrease in clay content, w_L and I_p also decrease. Regarding

mechanical properties for such diatomite mixtures, ϕ' and s_u/p increase with an increase in diatomite

content. It should also be noted that solid particles density (ρ_s) of diatoms is very small compared to

that of ordinary soil, around 2.7. 0 10 20 30 Counts per Second 500 N Angle 20 (°) H G Depth 15 m

0 10 20 30

Counts per Second 500 K Angle 20 (°) G N H Depth 6 m

0 10 20 30 H

Counts per Second 500 Angle 20 (°) G H Depth 22 m 0 10 20 30 Counts per Second 500 Angle 20 (°) G H N Depth 37 m (c) At 22 m depth (b) At 15 m depth. (d) At 37 m depth (a) At 6 m depth

Figure 7. Clay minerals measured by X-ray diffraction test.
 N: natural, G: treated by Glycolation, H: heated at 550 ° C

Figure 8. Microfabric of Hachirogata Clay using scanning electron microscope. Sample depth is 6 m. 0 100 200 20 15
 10 5 0 2.3 2.4 2.5 2.6 2.7 0 5 10 15 20 25 0 25 50 75 D e p
 t h (m) Water Content (%) w n (%) w L w P p s (g/cm
 3) Activity Diatom Content (%)

Figure 9. Influence of diatom contents on index properties, solid particles density and activity. Table 1. Comparison of Index properties of Hachirogata Clay before and after consolidation of 10 MPa. Sampling depth is 6 m depth. w L (%) w P (%) I p Before (intact) 243.9 67.0 176.9 After (10 MPa) 179.8 55.5 124.3

The ingredient of diatoms is mostly silica (SiO_2), which is sometimes called biogenic or amorphous

silica because of its non-crystalline structure. There are still controversies on how to quantify biogenic

silica, since some parts of non-biogenic silica derived from quartz etc., is also inevitably measured in

the process of quantifying total quantity of biogenic silica (see, Kamatani & Oku 1999). A preliminary

method has been developed for analyzing the amount of diatoms by Shiwakoti et al. (1999). A specimen,

weighing 0.6 g by dry weight, was treated with hydrogen peroxide and hydrochloric acid to remove

organic matter and disperse diatom cells from soil particles. Part of the solution was taken with a pipette

and mounted on a glass sheet. The specimen, thus prepared, was observed by optical microscope under

400 times magnification. The numbers of diatoms in a specified area were counted, from which the

total number of diatoms present per unit dry weight of soil was calculated. Of course, there are many

problems in evaluation of diatom numbers by this method. The size of diatom is not the same but varies

with species. Also some diatoms are not in the original shape, but are found broken into smaller pieces. In

this test procedure, such a fragment is counted as one. Therefore, the number of diatoms determined by

this method does not precisely indicate the volume of diatoms present in the soil specimen, nonetheless,

it provides at least a rough idea on whether a soil contains plenty of diatoms or not.

Based on knowledge of the influence of diatom presence from artificial mixtures of diatomite soils,

the physical properties of Hachirogata Clay were examined again. It can be seen in Fig. 9 that at shallow

depths, p_s is very small, whereas its activity is extremely large. It is very interesting in that at these

depths, diatom content is very large. From this evidence it may be concluded that such large index

properties are partly due to diatoms, in addition to two important facts: the clay mineral is smectite; and

high content of organic matter cannot be ignored for large index properties.

There is further indirect evidence of the existence of diatoms in the soil near the ground surface. Table 1

compares the Atterberg limits before and after consolidation at 10 MPa. The sample was recovered from 0 20 40 60 80 100 0 50 100 150 200 250 6m depth 16m depth S h e a r S t r e s s (k P a) Normal Stress (kPa) Hachirogata $\phi' = 40$ (6m) $\phi' = 33$ (16m) 250kPa 200kPa $p'_{vo} = 150$ kPa

Figure 10. Test result from direct shear test. A sample of 6 m depth contains a lot of diatom compared with that of

16 m depth.

a depth of 6 m. The w_L for a consolidated specimen decreases from 244% to 180% because the skeleton

has been crushed by the large consolidation pressure, so that the same quantity of water cannot be held

in its pores. Other evidence is the strength. Figure 10 shows a test result from the direct shear test for

specimens at depths of 6 m and 16 m. In this test, the specimen was sheared under constant volume, as

will be described in 6.1. From Fig. 9, it is known that a specimen at a depth of 16 m contains a smaller

amount of diatoms compared to that at 6 m. The value of ϕ' for a specimen at 6 m depth (40°) is much

higher than those at 16 m (33°). Also the peak undrained strength normalized by the p'_{vo} at 6 m is greater

than that at 16 m. This test result implies that the existence of diatoms creates higher ϕ' and the undrained

shear strength, which is the same result as that obtained in the artificially prepared diatomite mixture

(see Tanaka 2002; Shiwakoti et al. 2002).

5 CONSOLIDATION PROPERTIES

5.1 e-log p relation

Figure 11 shows e-log p relations measured by Constant Rate of Strain (CRS) tests, where a constant

strain rate of $0.02\%/min$ ($3.3 \times 10^{-6} s^{-1}$) was applied. In this test, the specimens were recovered by two

different samplers: the Japanese standard sampler with a stationary piston (JPN in the figure) and Shelby

tube without a piston. As revealed by Tanaka (2000), sample quality retrieved by the Shelby tube is con

siderably poorer compared with that by the Japanese sampler, evaluated by the unconfined compression

strength (q_u). It is apparent in Fig. 11 that the e-log p relations for the JPN sampler are located above

those for the Shelby tube. However, this difference is caused by sampling depth (the Shelby tube was used

at depths deeper than 12.8 m). Figure 12 compares the e-log p relations at nearly the same depth for JPN

and Shelby tube samplers. It may be concluded that the e -log p relation is not much influenced by sam

pling methods: i.e., both samplers present the similar e -log p relation, which indicates flat relation (low

compressibility) before the yield consolidation pressure (p_c) and sharp reduction of e after p_c . As will be

mentioned later, however, the q_u value is strongly affected by sampling method. This apparent inconsis

tency is also observed in other sites, for example, Ariake, where there was no notable difference in the

e -log p relations for different samplers including JPN and Shelby tube samplers, though there was a great

difference in the q_u values. However, for Bothkennar clay, poor quality samples provide small q_u values,

and the e -log p relations clearly are different from high quality samples (see Tanaka 2000 for more details). The p_c values are shown in Fig. 13, compared with the in situ effective overburden pressure (p'_{vo}). It is

well known that even in mechanically normally consolidated layers where the soil has not experienced

a larger consolidation pressure than the present p'_{vo} , the p_c value is somewhat greater than p'_{vo} . Bjerrum

(1973) insisted that such an apparent overconsolidation is caused by ageing and called such clay "aged

normally consolidated". For Hachirogata Clay, the apparent overconsolidation ratio (OCR) is rather

small, especially near the ground surface, but with the increase in depth, the OCR value increases to a

value of about 1.2. It is again confirmed in the figure that the p_c value is not influenced by sampling

methods. Figure 11 shows the e -log p relations at various depths. As consolidation pressure increases,

for example, more than 200 kPa, it seems that the e -log p relations concentrate into a single relation,

except for the specimen at a depth of 2.8 m. It is convenient to use relative void ratio for making a

comparison of the e -log p relations at different depths because index properties for these samples also

vary with depth. For such a purpose, several relative void ratios have been proposed, for example, 10 100 1000 0 1 2 3 4 5 6 V o i d R a t i o Consolidation Pressure (kPa) JPN 2.8 m 7.9 10.2 14.4 22.9 33.0 Shelby 12.8 18.8 24.8 30.9 38.9 42.8

Figure 11. e -log p relation measured by CRS test with 3.3×10^{-6} s $^{-1}$ strain rate. 10 100 1000 10000 0.8 1.2 1.6 2.0 2.4 2.8 3.2 V o i d R a t i o p' (kPa) JPN 22.7m JPN 33.0m Shelby 24.8m Shelby 30.9m

Figure 12. Selected e -log p relation. Samples were recovered by Japanese Standard sampler (JPN) and Shelby tube. 0 50 100 150 200 250 40 30 20 10 0 Measured by CRS (0.02%/min) p' v o D e p t h (m) p_c (kPa) JPN Selby

Figure 13. Yield consolidation pressure for Hachirogata Clay. 10 100 1000 -1.5 -1.0 -0.5 0.0 0.5 1.0 1.5 2.0 ICL, Burland, 1990 V o i d I n d e x Consolidation Pressure (kPa) JPN 2.8 m 7.9 10.2 14.4 22.9 33.0 Shelby 12.8 18.8 24.8 30.9 38.9 42.8

Figure 14. Relation with void index (I_v) and consolidation pressure.

Burland (1990); Tsuchida (2001), etc. The liquidity index I_L is one of the relative void ratios. Figure 14

shows the e -log p relation using Void Index (I_v), which was defined by Burland (1990) and calculated

using w_L . According to Burland's concept, as p increases and soil structure is destructed, then the I_v -log

p relation will be approaching a single line named "Intrinsic Consolidation Line" (ICL). It can be seen

in the figure, however, that most I_v s at these pressures are greater than those along the ICL, although

the slope of I_v -log p is almost the same as that of ICL. In the relation between I_v and log p , samples

obtained from shallow depths should be started from large I_v because of small p'_{vo} . However, as seen in

Fig. 14, the I_v value at the starting point for shallower depths is not always larger than that for samples

collected from great depths. Figure 15 shows the relation between I_{vo} and p'_{vo} , where I_{vo} is the in situ void index, calculated using

initial void ratio. In this figure, the relation between I_{vo} and p'_{vo} for other various sites are also plotted, in

addition to data from Hachirogata. Due to the fabric structure created in the process of sedimentation, it is
 100 1000 -1.0 -0.5 0.0 0.5 1.0 1.5 2.0 2.5 3.0 SCL,
 Burland, 1990 7 m Corresponding to 19 m ICL, Burland, 1990
 Void Index (Burland, 1990) p'_{vo} (kPa)
 Hachirogata Ariake Bothkennar Louiseville Singapore Bangkok
 Drammen Yamashita Pusan

Figure 15. Relation with initial void index (I_{vo}) and in situ effective overburden pressure (p'_{vo}) for Hachirogata Clay

as well as other clays from the author's investigation. 0
 50 100 150 200 250 300 0.0 0.5 1.0 1.5 2.0 2.5 C_{c1}
 $=0.009(w_L - 10) C_{c1} w_L$ (%) Hachirogata(JPN)
 Hachirogata(Shelby) Ariake Singapore Bangkok Yamashita
 Louiseville Bothkennar Drammen

Figure 16. Relation with w_L and C_{c1} , which is compression index at consolidation pressure large enough to be

constant.

is known that I_{vo} is somewhat larger than that along ICL. Burland (1990) accumulated data of the I_v -log

p'_{vo} relations for naturally deposited ground worldwide to draw a line of the relation between I_{vo} and log

p'_{vo} . He named this line "Sedimentation Compression Line" (SCL). For all soils except for Hachirogata

Clay, as p'_{vo} increases, I_{vo} decreases because of an increase in consolidation pressure. For Hachirogata

Clay, however, I_{vo} increases with increase in p'_{vo} and

reaches a peak at about p'_{vo} of 80 kPa (this pressure corresponds to 19 m depth), then I_{vo} decreases with increasing p'_{vo} . The I_{vo} at small p'_{vo} is lower than even that of ICL. This strange behavior may be due to high content of organic matter or microfossils, especially diatoms, as described before. When depth is greater than 19 m, the relation between I_{vo} and p'_{vo} joins the range of normal soils. This fact indicates that below a depth of 19 m, the influence of organic matter as well as diatoms is not so strong as to behave oddly.

5.2 Compression index

Compression index (C_c) has been correlated with index properties, especially w_L for many years. When

it is attempted to establish such a correlation, it should be noted that C_c is not a constant even in the 0 50 100 150 200 250 300 0 2 4 6 8 10 12 $C_{c1} = 0.009(w_L - 10)$ C_{cm} $a \times w_L$ (%) Hachirogata (JPN) Hachirogata(Shelby) Ariake Singapore Bangkok Yamashita Louiseville Bothkennar Yamashita

Figure 17. Relation with w_L and C_{cmax} , which is the maximum of C_c in the e -log p relation. 1 2 3 4 5 6 7 0 2 4 6 8 10 12 C_{cm} $a \times e_o$ Hachirogata(JPN) Hachirogata(Shelby) Ariake Singapore Bangkok Yamashita Louiseville Bothkennar Drammen

Figure 18. Relation with initial void ratio (e_o) and C_{cmax} , which is the maximum of C_c in the e -log p relation.

normally consolidated (NC) state as shown in Fig. 11, for example. When p exceeds the p_c value by

more than about 10 times, a constant C_c is usually obtained. If the C_c under these pressures is denoted by

C_{c1} , C_{c1} can be strongly correlated with w_L , as shown in Fig. 16. The relation of C_{c1} with w_L follows the

relation proposed by Terzaghi: $C_{c1} = 0.009(w_L - 10)$, including other soils accumulated by the Author's

investigations at various sites. On the other hand, the steepest gradient of the e -log p relation (C_{max}) cannot be correlated with w_L

(see Fig. 17). C_{max} is not related to the fundamental properties such as w_L , but probably has a strong

relation with structure. C_{max} is plotted against initial void ratio (e_0) in Fig. 18. Except for Louiseville

clay, the relation with e_0 and C_{max} is distributed in the same range, including Hachirogata Clay.

5.3 Hydraulic conductivity

For calculation of settlement, the rate as well as magnitude of settlement is important. It is known

that hydraulic conductivity (k) reduces with decrease in void ratio, and its reduction is expressed by $\log k = a - be$, where a is a constant. Figure 19 shows the relation between e and $\log k$ for Hachirogata Clay with various soils. Figure 19 shows the relation between e and $\log k$ for Hachirogata Clay with various soils. Figure 19 shows the relation between e and $\log k$ for Hachirogata Clay with various soils.

e	$\log k$	Soil
10	-12	Hachirogata
10	-11	Hachirogata
10	-10	Hachirogata
10	-9	Hachirogata
10	-8	Hachirogata
0.0	0.5	Osaka (Intact)
0.5	1.0	Osaka (Cons)
1.0	1.5	Bangkok
1.5	2.0	Bothkennar
2.0	2.5	Singapore
2.5	3.0	Drammen
3.0	3.5	London
3.5	4.0	Ariake
4.0	4.5	Pusan
4.5	5.0	Louiseville
5.0	5.5	Hachirogata (10.2m)
5.5	6.0	Hachirogata (18.8)
6.0	6.5	Hachirogata (22.9)
6.5	7.0	Hachirogata (30.9)
7.0	7.5	Hachirogata (38.9)
7.5	8.0	Hachirogata (42.8)

Figure 19. Comparison of hydraulic conductivity for Hachirogata Clay with various soils. Figure 19 shows the relation between e and $\log k$ for Hachirogata Clay with various soils. Figure 19 shows the relation between e and $\log k$ for Hachirogata Clay with various soils.

Figure 20. e -log p relation with different strain rate.

Clay measured by CRS test, in comparison with other clays. It can be seen that the location of e and

$\log k$ relation for Hachirogata Clay is in the upper band of the figure: i.e., Hachirogata Clay is rather

impermeable in spite of large e . This may be explained by the fact that, despite the abundance of

diatom skeletons having large porous structures, continuity of pores is rather poor, apparently due to the

considerable presence of a poorly permeable mineral like smectite.

5.4 Rate effect

It is known that since clay is a viscous material, the e -log p relation is influenced by the strain rate in

the same manner as strength. Figure 20 shows the e -log p curves measured by CRS tests under different

strain rates. As can be seen, when the strain rate is larger, the e -log p relation is shifted to the right. This

amount of shift is nearly proportional to the increase in the strain rate in logarithm scale. The ratio of

the p_c value due to an increase in the strain rate by 10 times is indicated in Fig. 21, where the range of

strain rate for the experiment is from 0.001%/min and 0.01%/min. Tanaka et al. (2000) tried to find out 1 1.05 1.1 1.15 1.2 1.25 0 50 100 150 Ariake Hachirogata Sakishima Kobe Yangsan Bothkennar Bangkok Drammen Louiseville r a t e e f f e c t o n p r a t e e f f e c t o n p c Plasticity Index, I_p

Figure 21. Rate effect on the p_c value for various soils (after Tanaka, et al., 2000).

factors influencing the change in the p_c value. However, there is no single factor governing the strain

rate effect on the e -log p relation. The only clear finding from their study was that the change in the p_c

value was not dependent on I_p , as shown in Fig. 21.

6 STRENGTH PROPERTIES

6.1 Undrained shear strength

Undrained shear strength (s_u) was measured by Field and Laboratory vane, unconfined compression and

direct shear tests. The laboratory vane test was carried out in the following manner. First, the soil sample

was extruded about 7 cm from the sampling tube, for index tests such as Atterberg limit tests. Then, a vane

with 20 mm in diameter and 40 mm high was inserted into the sample in the sampling tube (its diameter

was 75 mm). The rotation speed of the vane was $6^\circ/\text{min.}$, following the standard of JGS (see Tanaka 1994

for more detail). The size of specimens for the direct shear tests was 60 mm in diameter and 20 mm in

height. Specimens were reconsolidated to $p' = v_o$, then sheared at a shearing speed of 0.25 mm/min under

constant volume (equivalent to the undrained condition) (see Takada 1993). Unconfined compression

tests followed the standard of the JGS: i.e., the diameter and height of a specimen were 35 mm and

80 mm, respectively. The axial strain rate was 1%/min. The s_u values measured from these tests are plotted in Fig. 22. Laboratory tests (Unconfined com

pression, laboratory vane and direct shear tests) were carried out for samples with two different qualities,

i.e., retrieved by the Japanese standard sampler and the Shelby tube. From the unconfined compression

test, the value of $s_u (= q_u/2)$ for samples collected by the Shelby tube is remarkably lower than that by

the Japanese sampler. The s_u value from the Japanese sampler is larger than that from the Shelby tube

and nearly the same as that measured using the field vane test. However, the difference in the s_u values

between the two samplers is less significant in the measurement of the laboratory vane, and for the direct

shear test the difference in s_u cannot be identified. The reason for such difference in the s_u value for dif

ferent shear tests may be attributed to the amount of the residual effective stress (suction) remained in the

specimen. Tanaka (2000) has revealed that the losing residual effective stress does not necessarily cause

the breaking of soil structure. Whether the soil structure is disturbed or not can be judged by the shape

of the e -log p curve. It should be remembered that sample quality does not affect the e -log p curve as

shown in Figs. 11 and 12. Therefore, this suggests that the structure is not greatly damaged by sampling

even using the Shelby tube in the case of Hachirogata Clay. Although the suction of the specimen was

not measured in this investigation, it is inferred that the residual effective stress for samples obtained by

the Shelby tube is smaller than that by the Japanese sampler because of small unconfined compression

Depth (m)	Undrained Shear Strength (kPa)	Field Vane $q_u/2$ (JPN)	$q_u/2$ (Shelby)	Lab. Vane (JPN)	Lab. Vane (Shelby)	DS (JPN)	DS (Shelby)
0	10						
10	20						
20	30						
30	40						
40	50						
50	60						
60	70						
70	80						
80	50						
90	40						
100	30						
110	20						
120	10						

Figure 22. Comparison of undrained shear strengths measured by various shearing test for samples with different

sample quality.

Depth (m)	s_u/p'_{vo}	Field Vane $q_u/2$ (JPN)	$q_u/2$ (Shelby)	Lab. Vane (JPN)	Lab. Vane (Shelby)	DS (JPN)	DS (Shelby)
0	0.0						
0.1	0.1						
0.2	0.2						
0.3	0.3						
0.4	0.4						
0.5	0.5						
0.6	0.6						
0.7	0.7						
0.8	0.8						
1.0	1.0						
2.0	2.0						
3.0	3.0						
4.0	4.0						
5.0	5.0						
6.0	6.0						
7.0	7.0						
8.0	8.0						
9.0	9.0						
10.0	10.0						
11.0	11.0						
12.0	12.0						
13.0	13.0						
14.0	14.0						
15.0	15.0						
16.0	16.0						
17.0	17.0						
18.0	18.0						
19.0	19.0						
20.0	20.0						
21.0	21.0						
22.0	22.0						
23.0	23.0						
24.0	24.0						
25.0	25.0						
26.0	26.0						
27.0	27.0						
28.0	28.0						
29.0	29.0						
30.0	30.0						
31.0	31.0						
32.0	32.0						
33.0	33.0						
34.0	34.0						
35.0	35.0						
36.0	36.0						
37.0	37.0						
38.0	38.0						
39.0	39.0						
40.0	40.0						

Figure 23. Normalized undrained shear strength by the in situ effective overburden pressure.

strength (q_u). However, it can be assumed that the soil structure is not significantly distracted since

the same s_u values are measured for the Shelby tube, when the specimen is consolidated under p'_{vo} in

the direct shear box. In the case of the laboratory vane, some part of the residual effective stress and

confining stress remained even in the Shelby tube sample, because the specimen was confined by the

sampling tube.

Figure 23 shows the undrained shear strength normalized by p'_{vo} . The influence of sample quality on

the shear strength is much more prominent than that in Fig. 22. It can be seen in the figure that near the

ground surface, the s_u / p'_{vo} is very large, although the p_c value is the same as p'_{vo} (see Fig. 13). The depths

indicating large s_u / p'_{vo} also correspond to those where a lot of organic matter as well as diatoms are found.

It is usually observed even in normal ground that the s_u / p'_{vo} ratio near the ground surface is relatively

large. Such a large s_u / p'_{vo} is mainly attributed to high OCR created by fluctuation of the ground water

table. However, as seen in Fig. 13, the OCR near the ground surface is not large enough to explain the

high s_u / p'_{vo} . Therefore, it is inferred that the large s_u / p'_{vo} values may be partly attributed to the presence

of organic matter or diatoms. However, below a depth of 15 m, the s_u / p'_{vo} ratio becomes constant and

its value lies in the range of 0.3 to 0.34, excluding the strengths influenced by poor quality samples. As

mentioned before, the OCR measure by CRS tests shows 1.2 for moderate depths. Consequently, the 0.1 0.2 0.3 0.4 0.5 0.6 0.20 0.40 0.60 0.80 1.00 Japanese clays at 7 sites Bothkennar Drammen Singapore Louiseville Swedish Clays from Larsson s_u / p_y (from Field Vane) Plasticity Index, I_p $s_u / p_c = 0.11 + 0.0037 I_p$ (Skempton) Pusan

Figure 24. Relation between Strength incremental ratio and I_p . Data for Swedish clay is referred after Larsson

(1991).

strength increment for consolidation pressure (s_u / p_c) is about 0.27. Tanaka (2000) has revealed that s_u / p_c

measured by the field vane for Japanese clays is independent of I_p and relatively high compared with

other clays, as shown in Fig. 24. Tanaka & Locat (1999) and Shiwakoti et al. (1999) have pointed out that

one of the reasons for high s_u / p_c value for Japanese clays is the high content of microfossiles, especially

diatoms.

6.2 Cone factor for CPT

Figure 25 correlates the net resistance ($q_t - p_{vo}$) from CPT with s_u from the unconfined compression

and the field vane tests, where q_t and p_{vo} are, respectively, point resistance considering the effective area

of the cone and the in situ total overburden pressure. The cone factor, N_{kt} ($s_u = (q_t - p_{vo})/N_{kt}$)

ranges from 8 to 10, as indicated in Fig. 25. Although there is controversy as to whether N_{kt} is dependent

on I_p , N_{kt} for Japanese clays seem to be independent of I_p , and range from 9 to 14, based on s_u from the

field vane test (see Fig. 26).

6.3 Bjerrum's correction factor

Bjerrum's correction factor is widely used to modify s_u from the field vane test for application of the

undrained shear strength to the total stress analysis for stability. On the other hand, Japan has traditionally

used s_u evaluated from the unconfined compression test ($q_u/2$) without any correction factor. Figure 27

shows comparison between $q_u/2$ and μs_u , where μ is Bjerrum's correction factor determined by I_p

(Bjerrum, 1973) and where s_u is measured by the field vane test. It is clearly observed in the figure

that the ratio of $\mu s_u(\text{vane})/(q_u/2)$ is dependent on I_p : i.e., at relatively small I_p , $\mu s_u(\text{vane})$ is larger than $q_u/2$

(i.e., $\mu s_u(\text{vane})/(q_u/2)$ ratio is greater than 1.0), while $q_u/2$ is larger when I_p exceeds 40. The reason for

large $\mu s_u(\text{vane})$ values may be due to underestimation of the s_u obtained by $q_u/2$. As already mentioned,

Japanese clay consists mainly of clay minerals with high activity such as smectite. Therefore, soil with

low I_p contains a lot of coarse materials such as silt or sand, consequently the suction in such soils cannot

be retained when the soil is sampled and exposed to the atmosphere. The q_u value is strongly affected

by the amount of the residual effective stress (suction) because the test is carried out under unconfined

stress conditions. Therefore, it can be judged that the $q_u/2$ value for low I_p underestimates the undrained

shear strength. It is interesting that for overseas clays (shown by open symbols in Fig. 27), which present $0 \leq D_p \leq 20$ $30 \leq D_p \leq 40$ $25 \leq D_p \leq 30$ $15 \leq D_p \leq 25$ $10 \leq D_p \leq 15$ $5 \leq D_p \leq 10$ $0 \leq D_p \leq 5$ $N_{kt} = 10$ $N_{kt} = 8$ D_p (m) Undrained Shear Strength (kPa) $q_u/2$ Vane

Figure 25. Cone factor for Hachirogata Clay. $0 \leq D_p \leq 20$ $20 \leq D_p \leq 40$ $40 \leq D_p \leq 60$ $60 \leq D_p \leq 80$ $80 \leq D_p \leq 100$ $0 \leq D_p \leq 2$ $2 \leq D_p \leq 4$ $4 \leq D_p \leq 6$ $6 \leq D_p \leq 8$ $8 \leq D_p \leq 10$ $10 \leq D_p \leq 12$ $12 \leq D_p \leq 14$ $14 \leq D_p \leq 16$ N_{kt} (based on Vane) I_p Kurihama Izumo Kuwana Ohgishima Shinonome Tamano

Figure 26. Cone factor for various Japanese clays.

relatively low I_p even with large contents of clay particles, their scatter in $\mu_s u(\text{vane}) / (q_u/2)$ is small and

their ratio is generally less than 1.0.

On the other hand, the $q_u/2$ is considerably larger than $\mu_s u(\text{vane})$ at moderate and large I_p (and for most

overseas clays). This leads to the question “Does the unconfined compression test overestimate the true

strength?” If the field vane strength modified by Bjerrum’s correction factor provides the true strength

and the $q_u/2$ gives unsuitable strengths for stabilization analysis, many accidents of failures should

have been reported because foundations are designed based on $q_u/2$ value in Japan, as mentioned before.

However, such reports of such cases have never been published, neither has there been any reconsideration

to change the design method using $q_u/2$. Rather, it should be concluded that the modification by the

Bjerrum's correction factor gives conservative s_u for foundation design, at least for Japanese soils. Taking

a look at overseas cases, for example, as shown in Fig. 27, $\mu s_u(\text{vane})$ for Bothkennar, Pusan and Singapore

clays is considerably smaller than their $q_u/2$ value even at small I_p . Although it is true that only Japan

evaluates the undrained shear strength by the unconfined compression test, Bjerrum's correction factor

probably underestimates the undrained shear strength. The concept of the Bjerrum's correction factor

is derived for modifying the strain rate and the anisotropy effects: i.e., the rotation speed of the field θ 20 40 60 80 100 120 140 160 0.0 0.5 1.0 1.5 $\mu s_u(\text{vane}) / (q_u/2) I_p$ Tamano Shinonome Kurihama Hachirogata Izumo Kuwana Ohgishima Singapore Bothkennar Drammen Pusan Louiseville

Figure 27. Comparison of undrained shear strength for design: Bjerrum's and q_u methods. θ 20 40 60 80 100 120 0.4 0.6 0.8 1.0 1.2 1.4 Hachirogata Bothkennar Louiseville Bangkok Pusan Yamashita Drammen Singapore Ariake Osaka Anisotropy, s_{ue}/s_{uc} I_p

Figure 28. Anisotropy of undrained shear strengths. s_{ue} and s_{uc} are from extension and compression triaxial tests, respectively.

vane test is too fast compared with that in the field, and shear strength derived from the field vane test is

mainly mobilized along the vertical plane. Bjerrum considered that these effects are simply correlated

with I_p and he proposed a single line, which is derived based on I_p for modification of the vane strength.

However, as already indicated in Fig. 21, the rate effect on the e - $\log p$ relation cannot be correlated with

I_p : i.e., the rate effect does not increase with increase in I_p , unlike his assumption for deriving μ . Figure 28 shows anisotropy in the undrained shear strength as the ratio of s_{ue}/s_{uc} , where s_{ue} and

s_{uc} are the peak strength from extension and compression triaxial tests, respectively. The specimens in

these tests were consolidated under pressures large enough to exceed their p_c value and following K_o

conditions. Although a tendency for s_{ue}/s_{uc} to increase with I_p can be slightly recognized, there is an

explicit exception in these relations, i.e., Singapore clay. For this clay, the extension strength is larger

than compression one. It is recommended that before Bjerrum's correction factor is applied to sites without experiences, its

validity should be examined carefully. 0 5 10 15 20 25 30 40 30 20 10 0 50($q_t - p_{vo}$) Tanaka et al., (1994) Depth (m) G_{sc} (MPa)

Figure 29. Shear modulus measured by seismic cone, compared with estimated value from CPT (Tanaka et al.,

1994). 0 1 2 3 4 5 6 0 100 200 300 400 500 600 G_{sc}/p'_{vo} e o Hachirogata Other Japanese Clays Singapore Pusan Louiseville Bothkennar from Nash et al. Bangkok Scandinavian from Larson

Figure 30. Ratio of shear modulus to the effective overburden pressure versus void ratio.

7 SHEAR MODULUS MEASURED BY SEISMIC CONE

Shear modulus for Hachirogata was measured by a seismic cone (G_{sc}). The result is shown in Fig. 29,

together with estimated G_{sc} from the CPT using an empirical relation $G_{sc} = 50(q_t - p_v)$ (Tanaka et al.,

1994). From the figure, the estimated G_{sc} is somewhat larger than the measured one.

Until now, many researchers have investigated what factors govern the G_{sc} value. Figure 30 shows

the relation between G_{sc}/p'_{vo} and in situ void ratio (e_o). Although the variation in G_{sc}/p'_{vo} becomes large 0 500 1000 1500 2000 2500 0.5 1.0 1.5 2.0 2.5 3.0 3.5 4.0 4.5 G_{sc}/s_u (vane) e o Hachirogata Other Japanese Clay Louiseville Bothkennar from Nash et al. Singapore

Figure 31. Ratio of shear modulus to the vane shear strength versus void ratio. 0 25 50 75 100 125 150 175 0 500 1000 1500 2000 2500 $G_{sc} / s_u (vane) I_p$
Hachirogata Other Japanese Clays Louiseville Bothkennar from Nash et al. Singapore Scandinavian from Larson Pusan

Figure 32. Ratio of shear modulus to the vane shear strength versus plasticity index.

at small e_o , especially less than 2.5, there exists an explicit tendency: with an increase in e_o , G_{sc} / p'_{vo}

becomes smaller. Especially for Japanese clays including Hachirogata Clay, the relation between G_{sc} / p'_{vo}

and e_o is clearly observed. The reason for the large scatter of G_{sc} / p'_{vo} for overseas clays is not clearly

identified; however, one of the key factors for such large scatter is variation in OCR. It is reasonable

to consider that as OCR increases, G_{sc} / p'_{vo} should increase if other conditions are identical. In this

sense, the normalization of G_{sc} by s_u measured from the Field vane test, for example, seems much more

reasonable than that by p'_{vo} , because s_u is also influenced by OCR. The relation between $G_{sc} / s_u (vane)$ is

shown in Fig. 31, plotted against e_o . Compared to that in Fig. 30, the scatter at lower e_o in $G_{sc} / s_u (vane)$ is

much smaller than in G_{sc} / p'_{vo} , and the relation for overseas as well as Japanese clays converges within

a relatively narrow band. When e_o is larger than 2.5, the $G_{sc} / s_u (vane)$ slightly decreases with e_o . On the

other hand, at e_o smaller than 2.5, $G_{sc} / s_u (vane)$ considerably increases with decrease in e_o . Figure 32 plots relation of the $G_{sc} / s_u (vane)$ against I_p , instead of e_o , because e_o is not an inherent value,

but is influenced by consolidation pressure, i.e., depth or OCR. However, the appearances of these two

figures are not much different regardless of whether e_o or

I_p is plotted. Since the depths at the sites used for these figures are somewhat restricted (the maximum depth is at most 45 m), e_o and I_p are strongly related. That is, if I_p is large, e_o is also inevitably large. Nevertheless, the degree of scatter in $G_{sc}/s_u(\text{vane})$ against I_p is much smaller at large I_p . However, at small I_p , the data scatters in both Figs. 31 and 32 are almost identical.

8 CONCLUSIONS

From a series of site investigations at the Hachirogata site, where thick clay with extremely large plasticity

Index (I_p) is deposited, the following interesting facts are found.

- 1) It is inferred that the reasons for large I_p of Hachirogata Clay are: clay mineral with large activity such as smectite; high content of organic matter and a large amount of macrofossils, especially diatoms.
- 2) Two different sample qualities were prepared, i.e., using a Japanese standard sampler with a stationary piston and a Shelby tube without a piston. The e -log p curves for soils with two different sample qualities are nearly identical. However, there is a large difference in the unconfined compression strength (q_u) between the Japanese sampler and Shelby tube. This difference may be caused by difference in the residual effective stress (suction) in samples collected by different samplers.
- 3) The yield consolidation pressure (p_c) is slightly larger than the in situ effective overburden pressure (p'_{vo}). At moderate depths, the overconsolidation (OCR) ratio is about 1.2.
- 4) The compression index at large consolidation pressure (C

c_1) can be well correlated with liquid limit

(w_L) : $C_{c1} = 0.009(w_L - 10)$.

5) Undrained shear strength normalized by p'_{vo} (s_u/p'_{vo}) near the ground surface is extremely large even

though the soil is not so overconsolidated. This large s_u/p'_{vo} is inferred to be partly attributed to a large

amount of organic matter and diatoms. At greater depth where the influence of organic matter and

diatoms is insignificant, the s_u/p'_{vo} reduces to a more usual value, i.e., around 0.32.

6) The cone factor for piezocone (N_{kt}) is between 8 and 10, this range falls within the value of 9 to 13

for Japanese marine clays.

7) Bjerrum's correction factor provides conservative undrained shear strength, compared with the uncon-

fined compression strength, which is extensively used in Japan. It is recommended that before

Bjerrum's correction factor is applied to sites without experience, its validity should be examined

carefully.

8) The shear modulus obtained by the seismic cone (G_{sc}) is normalized by p'_{vo} or s_u from the vane test,

(G_{sc}/p'_{vo}) or $(G_{sc}/s_u(\text{vane}))$. Both G_{sc}/p'_{vo} and $G_{sc}/s_u(\text{vane})$ are dependent on the in situ void ratio (e_o).

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Depositional geochemistry and geotechnical properties of marine clays in the Ariake Bay area, Japan

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ABSTRACT: The present paper provides integrated information on the geological setting, the miner

alogy, the depositional and post-depositional geochemistry, and the overall geotechnical properties for

Ariake Bay sediment based on the data of the borehole samples collected from different sites in Ariake

Bay area. The variation of the geotechnical properties of the sediment, including consistency limits, shear

strength, sensitivity and compressibility, can be explained in terms of clay mineral composition, pore

water chemistry, and chemical changes of soil materials due to weathering. There exist differences in

the geotechnical properties between salt-leached and non-salt-leached clay sediment. The low swelling

nature of the smectite which is the principle clay mineral of the sediment contributes to the formation

of quick clay in salt-leached sediment.

1 INTRODUCTION

Recent marine deposits fall into two groups, those with glacial and non-glacial origins, according to

the depositional environment of the deposits. Glacial clay sediments of ground rock flour produced by

the grinding action of the ice are found in Scandinavia and Eastern Canada (Bjerrum 1967, Quigley

1980, Locat et al. 1984). These sediments consist predominantly of mica, chlorite, quartz, and feldspars;

high swelling minerals like smectite have not been detected. After glacial retreat the sediments had been

elevated above sea level and salt in the pore water has been removed by leaching, which led to the

development of quick clay (Rosenqvist 1953, Bjerrum 1954, Torrance 1975, Locat & Lefebvre 1985).

Quick clay exhibits liquid behavior upon remolding due to an increase in the interparticle repulsive force

followed by a reduction in pore water salinity. Reduction in the remolded strength is attributed to the

predominance of low swelling minerals in the clays (Torrance 1987). The development of quick clay has

led to landslides in the clay sediments of Scandinavia and Eastern Canada.

Alluvial deposits, which are typical in recent marine sediments with non-glacial origin, are widely

found in South East Asian countries (Cox 1968). Alluvial deposits can be divided into two groups,

those with pyroclastic and non-pyroclastic origin according to the parent material of the deposits. In the

coastal areas of Japan, alluvial marine deposits with pyroclastic origin are widely distributed. Egashira &

Ohtsubo (1984) and Ohtsubo et al. (1999) examined the clay mineralogy of the deposit samples collected

from different sites in Japan, and found that smectite is a main clay mineral. The authors assume that the

smectite in the deposits is an in-situ formation product from pyroclastic material, since the watershed of

the rivers flowing into waters where smectitic marine clays are deposited, is mostly covered with volcanic

ash. Predominance of smectite is reflected in the geotechnical properties of Japanese marine clays such

as rather high liquid limit and activity (Tanaka et al. 2001, Fujikawa & Takayama 1980a). Presence of

diatom in greater amount in the deposits is also one of the significant features of Japanese marine clays,

which could enhance the liquid limit, friction angle, and undrained shear strength of the clays (Tanaka

2002). Alluvial marine deposits with non-pyroclastic origin can found in the countries other than Japan,

and the mineralogy and geotechnical properties of the deposits have been studied (Ohtsubo et al. 2000,

Tanaka et al. 2001, Ohtsubo et al. 2002). Bangkok clay consists predominantly of smectite, and the

liquid limit and activity are as high as Japanese marine clays. Pusan clay in Korea consists mainly of

kaolinite, illite, and vermiculite, and Singapore clay has kaolinite. Predominance of these inactive clay

minerals gives rather low liquid limit and activity compared to Japanese clays. Singapore marine clay

gave smaller friction angle compared to Ariake clay. Sensitive marine clays with pyroclastic origin 15 to 40 m thick extend in the coastal plain around

Ariake Bay in Kyushu, Japan. Ariake clay is the alluvial deposits of fine soil particles transported by the

rivers flowing into the bay from the watershed widely covered with volcanic ash soil. Fossil assemblage

analysis of the sediment indicated that the deeper zone below approximately 11 m was deposited under

rapid transgression; the zone above 11 m was deposited under relatively stable marine conditions (Ariake

Bay Research Group 1965). Analysis of the fossil of shells indicated that the surface zone of the sediment

was deposited in fresh or brackish conditions, the middle zone in marine conditions and the deeper zone

in brackish conditions (Shimoyama et al. 1994). Such geological history of the clay sediment has been

noted to reflect in the depositional geochemistry during sedimentation and post depositional processes

(Torrance & Ohtsubo 1995, Ohtsubo et al. 1995). Ariake clay sediments contain smectite as a principle clay mineral (Aomine et al. 1954, Egashira &

Ohtsubo 1982, Ohtsubo et al. 1995). The predominance of smectite characterizes the geotechnical index

properties of the clay deposits; their liquid limit and activity are in the range of 50 to 140% and 0.7 to

2.3, respectively (Fujikawa & Takayama 1980a), which are greater compared to those of marine clays in

Scandinavia and Eastern Canada and those of marine clays in South East Asian countries. Laboratory

tests on Ariake clay samples indicated that the liquid limit and remolded strength were decreased by salt

leaching, leading to the formation of quick clay (Ohtsubo et al. 1982, Miura et al. 1997). The response

of the liquid limit and remolded shear strength against pore water salinity for Ariake clay was contrary

to that normally associated with high-swelling smectite clay like bentonite, but rather similar to that for

clays composed of low-swelling minerals in Scandinavia and Eastern Canada. Such unusual behavior of

Ariake clay was found to be due to the low-swelling nature of the smectite in Ariake clay (Egashira &

Ohtsubo 1981). Similar behavior in the liquid limit and remolded strength was observed in marine clays

in Japan other than Ariake clay, and the smectite in the clays was also found to be a low-swelling type

(Ohtsubo et al. 1999). Ariake clay sediments, because of high sensitivity and compressibility, have posed several geotechnical

problems such as failure of slopes in irrigation canals, the settlement of road and earth structures (Miura

et al. 1988), and ground subsidence due to excess pumping up of ground water (Hiwatashi & Iwao

1975, Miura et al. 1986). To tackle these problems, numerous studies on the geotechnical properties of Ariake clay deposits have been made. Sridharan et al. (2000) re-examined the physical, physico-chemical and the engineering behavior of Ariake clay with a primary focus on its grouping as a non-swelling or swelling soil. Physical properties of Ariake clay from different sites were summarized and compared with those of Japanese marine clays other than Ariake clay (Fujikawa & Takayama 1980a). Nakamura et al. (1988) investigated the overall geotechnical properties of Ariake clay. Miura et al. (1997) examined the geotechnical properties of clays and found that quick clay formation is a result of salt removal by leaching followed by excess pumping of ground water. Li et al. (1998) attempted to describe the geotechnical characteristics of marine and non-marine sediment based on a data-base system focusing on the depositional environment. The geotechnical properties of Ariake clay were compared with those of Singapore and Bangkok clay (Tanaka et al. 2001). Fujikawa & Takayama (1980c) studied on the ratio of the undrained shear strength to the consolidation pressure of natural clays, and indicated that the change in this ratio can be explained in terms of the geotechnical indexes such as plasticity index, liquidity index and sensitivity. Strength characteristics of undisturbed Ariake clay in the inner area of Isahaya Bay were studied based on the results of constant volume direct shear tests and isotropically consolidated undrained triaxial compression tests (Higashi et al. 2000). Onitsuka et al. (1976) and Higashi & Takayama (1985) investigated the strength anisotropy of Ariake clay deposits. Stability analysis of slopes has

been performed taking into account of strength

anisotropy of clay (Fujikawa et al. 1987).

Yamadera et al. (1998) discussed cementation effects on the compression and unconfined shear

strength characteristics of undisturbed Ariake clay. Hanzawa et al. (1990) determined undrained shear

strength and compressibility of Ariake clay using various field and laboratory tests, and discussed the

causes of overconsolidation of the clays. Hong & Onitsuka (1998) proposed a method of correcting yield stress and compression index of Ariake

clays for sample disturbance. Hong & Tsuchida (1999) studied the compression behavior of Ariake clay

using the concept of void index proposed by Burland. Analytical and experimental investigation was made

on compression index equation for undisturbed Ariake clay (Park & Koumoto 1998). One dimensional test

and isotropic consolidation tests were conducted using a triaxial compression apparatus for undisturbed

Ariake clay to determine the coefficient of earth pressure at rest, and the compression and swelling

index (Takayama et al. 2000). The characteristics of the coefficient of secondary consolidation for clay

minerals, and undisturbed and remolded Ariake clays were examined using standard consolidation tests

(Kanayama et al. 2001).

The present work was undertaken to provide integrated information on the geological setting, the

mineralogy, the depositional and post-depositional geochemistry, and the overall geotechnical properties

of Ariake Bay sediment based on the data of the borehole samples collected from different sites in Ariake

Bay area. The observed variations in the geochemistry of

the clay profile with depth are interpreted

in terms of depositional and post-depositional processes.
The geotechnical characteristics of Ariake

clay including physical properties, shear strength,
sensitivity, and compressibility are discussed taking

account of the mineralogy and geochemistry of the sediment.

2 INITIAL CONDITIONS

2.1 Geological setting

Ariake Bay occupies an area of approximately 1600 km² on
the western side of Kyushu (Fig. 1). Its

outlet is approximately 5 km wide. The current sea-water
depth is 20 m at high tide over most of the

area, although greater depths occur in the southern portion
near the outlet. The tidal variation in the

Figure 1. Ariake Bay.

Figure 2. Sea-level movement curves for a period during the
past 9000 years in Fukuoka City (Shimoyama et al.

1991).

bay during spring tide is as high as 6 m, which has
promoted projects for reclamation by drainage to

create agricultural land over the past 60 years. Ariake Bay
sediment, with its surface now above sea level

(altitude <5 m), covers several hundred square kilometers
at the north end of the bay, and lesser areas in

other locations around its perimeter. The sea-level
movement curve over the past 10,000 years in the Fukuoka
area 50 km from the bay is

shown in Figure 2 (Shimoyama et al. 1991). The curve is
based on ¹⁴C (carbon 14)-dating and paleoenvi

ronmental analysis of molluscan fossils. A rapid
transgression, called the Jomon Transgression in Japan,

was observed 10,000-6000 years ago, which is equivalent to

the worldwide Flandorian Transgression.

The highest sea level occurred at 2-3 m above the present sea level about 5000 years ago. A small-scale

regression was noted 3000-2000 years ago. A sea-level movement curve is represented as the sum of

eustatic sea-level and local ground movement (Walcott 1972). The ground settlement in the Fukuoka area

during the period of the highest sea level was estimated to be 2 m from data on the regional differences

in the upper limit of marine deposits determined from the paleoenvironmental analysis of molluscan

fossils (Shimoyama et al. 1991). The magnitude of the ground settlement thus estimated is included in

the sea level movement curve in Figure 2. Torrance and Ohtsubo (1995) reconstructed the recent history of Ariake Bay, based on the depth and

width of its outlet and sea-level changes over the past years. Before 12,500 years ago, the sea level was

below 50 m (the depth of the outlet) and the deeper portions of Ariake Bay would have contained fresh

water. As the sea level rose and Ariake Bay became tidal, the salinity of the basin would have been

determined by the balance between the input of fresh water from rivers and the entry of marine water

by tidal action and density flows. As tidal currents sweep into the bay, they move northwards along the

eastern shoreline and create a counterclockwise water movement. This would sweep the finer suspended

particles delivered by rivers on the east side towards the inland end, where sedimentation would occur.

During the Jomon Transgression the width of the bay's outlet would have been less than 5 km of the

present outlet. This would have restricted the amount of sea water entering the bay, so that brackish

conditions might be expected to have been obtained throughout most of this period, particularly in the

northern reaches, which are about 100 km from the outlet. During this period, the surface zone of the clay

sediment would have been under conditions in which the clay material in the surface zone was frequently

exposed to a subaerial environment, resulting in weathering of the clay material due to oxidation. At

some point during the rising of the sea level, the entry of sea water would have been sufficient for the

salinity to approach the current near-marine conditions. The lowering of the sea level after its peak would

then have again created brackish conditions. Figure 3 shows the geological profile of the clay deposits along the line of BB' in Figure 1. (Shimoyama,

et al. 1994). The Holocene clay deposits (Hasuiki and Ariake stratum) that we discuss in this paper

comprise soil materials sedimented under marine or brackish conditions. The depositional environment

of the clay was assessed in terms of whether shells are present in the deposits or not. That is, the shells

are hardly dissolved in marine conditions since the pH of genuine sea water is 8.3 due to super saturation

of calcium and bicarbonate ions while the shells are dissolved in fresh or brackish conditions because of

their weak acid below pH 7. The thickness of the Holocene clay deposits increases from 10 m at a location

of 3 km to 30 m at a location of 20 km toward Ariake Bay. Non-marine deposits (Hasuiki stratum) are

predominant in a location of 3 to 7 km. In a location of 7 to 17.5 km the non-marine deposits are present

Figure 3. Geological profile of Ariake clay deposits (Shimoyama et al. 1991).

in the upper and deeper zone with marine deposits (Ariake stratum) sandwiched between the two zones.

Beyond 17.5 km only marine deposits are distributed.

2.2 Mineralogy

2.2.1 Parent materials

Figure 4 shows the area soil map around the rivers flowing into Ariake Bay. Red-yellow and brown forest

soils derived from granitic rock are widely distributed in the mountainous area along the Rokkaku and

Kase rivers. Andosol derived from pyroclastic material is predominant in the mountainous area along

the rivers (Chikugo, Kikuchi, Shira and Midori) in the eastern area. Grey lowland soils distributed in

flat areas along the rivers has been referred to as Ariake Bay sediment. Most of the sediment is the

accumulated product of the red-yellow and brown forest soils and andosol in sea water which have been

transported by the rivers. Trace element analyses (samarium and zirconium) have determined that the

sediments are dominated by soil materials of granitic origin from the base to approximately 13 m depth,

and pyroclastic material increases in abundance at lesser depths (Ohtsubo et al. 1995).

Figure 4. Soil map of the area around the rivers flowing into Ariake Bay: brown forest soil and red-yellow soil are

equivalent to dystrophic cambisol, and chromic cambisol or orthic Acrisol respectively on an FAO/UNESCO world soil

map (Ohtsubo et al. 1995).

2.2.2 Clay mineralogy

Figure 5 shows the X-ray diffraction patterns of magnesium clay specimens (<2 μm) solvated with

glycerol for samples from different depths (0.9 to 18.4 m)

obtained in Ariake Town, Saga Prefecture. The

strong peaks at 1.8 nm (18 Å) for all the depths indicate the presence of considerable smectite. Other

clay minerals identified were illite, vermiculite, chlorite, kaolinite and halloysite (1 nm). The silt and

fine-sand fractions contained quartz, feldspars and cristobalite. The percentage of the respective clay minerals in clay fractions was estimated based on the peak areas of

the clay minerals in the X-ray diffraction patterns (Egashira, et al. 1999, Ohtsubo et al. 2002). The results

obtained are shown in Figure 6, where the percentage of the respective clay minerals is plotted against

depth. The percentage was highest for smectite, followed by illite, above 14 m depth. The smectite content

was 23-24% at a depth of 14-18 m, increased upwards to a maximum of 60% at 3 m, and then decreased

slightly towards the surface. Halloysite (1 nm), which is the main clay mineral of weathered granitic

rocks, exhibited a vertical distribution opposite to that for smectite. Cole & Shaw (1983) indicated that

the alteration in the pyroclastic material was one of the principal modes of authigenic smectite formation

in recent marine sediment. Given that smectite content in the deposits increased towards the surface with

the increasing concentration of soil material of pyroclastic origin, the smectite in the Ariake sediment

would be an in situ formation product from the pyroclastic material.

2.2.3 Low swelling nature of the smectite

Smectite normally exhibits high-swelling characteristics, and hence the liquid limit of smectitic clay

is changed by changing pore water salinity through alteration of the thickness of the diffuse double

layer. The liquid limit of montmorillonite (smectite) with high-swelling characteristics for different pore

water salinity is shown in Figure 7. The liquid limit of Na-saturated clay decreased with increasing salt

concentration and by substituting Ca for Na. In the high-swelling smectite saturated with Na, intra

crystalline swelling arises from osmotic activity of the diffuse ion-layer around the unit layers of the

smectite (Warkentin 1961). With increasing salt concentration or substituting Ca for Na, intra-crystalline

Figure 5. X-ray diffraction patterns of the magnesium

clay solvated with glycerol (Ohtsubo et al. 1995). 0 5 10 15 20 20 40 60 Smectite Illite Halloysite Chlorite Kaolinite Vermiculite Clay mineral percentage: % Depth : m Figure 6. Variation of the clay mineral percentage with depth (Ohtsubo et al. 1995). 0.0001 0.001 0.01 0.1 1 200 400 600 800 1000 Salt concentration (mol c /L) Liquid limit (%) Na-clay Ca-clay

Figure 7. Liquid limit of montmorillonite as a function of salt concentration (Warkentin 1961).

swelling is decreased and the particles become free to move at lower water contents or lower interparticle

distance, resulting in the decrease in the liquid limit for the montmorillonite (Fig. 7).

The liquid limit of Ariake clay is shown against pore water salinity in Figure 8. In contrast to montmo

rillonite, the liquid limit of the Ariake clay sample (E-17) saturated with Na increased with increasing salt

concentration, and with substitution of Ca for Na in the whole concentration range. Thus the response

of the liquid limit to the salt concentration and cation species was opposite to that of montmorillonite

with high swelling nature. This unusual behavior was found to be due to the low-swelling characteristics

of smectite in Ariake clay (Egashira & Ohtsubo 1981). The change in the liquid limit in Figure 8 can be

explained by considering the interparticle net force of Van der Waals attraction minus electric repulsive

force (Rand & Melton 1977). Increasing the salt concentration and the valence of the exchangeable ions

decreases electric repulsion while leaving the force of attraction unchanged, resulting in an increase in θ .
 0.001 0.01 0.1 1 80 100 120 140 160 180 Salt concentration (mol c /L) L i q u i d l i m i t (%) Na-soil Ca-soil

Figure 8. Liquid limit of Ariake clay with low-swelling smectite as a function of salt concentration. Table 1. Sediment volume of the $<2 \mu\text{m}$ clay fractions at 0.04 mol c /kg (N) salt concentration. Na-clay Ca-clay (a) (b) Ratio Sample Cm 3 /100 mg Cm 3 /100 mg (a/b) Yamaashi Y-2 3.08 2.97 1.03 Y-6 2.63 2.69 0.98 Y-8 2.49 2.50 1.00 Y-13 2.43 2.37 1.02 Y-15 2.41 2.37 1.02 Higashi S-3 2.56 2.93 0.87 -shiroishi S-6 2.50 2.66 0.93 Saga paddy W-179-3 4.10 2.58 1.59 soil Wyoming 15.60 5.39 2.89 montmorillonite

the liquid limit (Fig. 8). Similar changes in the liquid limit with pore water salinity were also observed

for Japanese marine clays other than Ariake clay (Ohtsubo et al. 1999). The swelling characteristics of smectite in Ariake clays were assessed by the difference in the sed

iment volume between Naand Ca-clays (Egashira & Ohtsubo 1982). Table 1 shows the sediment

volumes of Naand Ca-clays and the ratio of the sediment volume of Na-clay to Ca-clay for Ariake clays

obtained at Yamaashi and Higashi-shiroishi, along with the results for Saga paddy soil (W-179-3) and

Wyoming montmorillonite. Saga paddy soil and Wyoming montmorillonite were employed as reference

samples since the smectite in those samples is of high-swelling type. For Saga paddy soil and Wyoming

montmorillonite, the sediment volume was greater for Na-clay than for Ca-clay, which indicates that

smectite in these materials is high-swelling type. On the

other hand, the sediment volume for Ariake clay

exhibited almost the same values for Na and Ca-clay, indicating that the smectite in Ariake clay is low

swelling type. The low-swelling smectite was considered to be a beidellite-nontronite mineral, and the

low-swelling characteristics of this smectite have been ascribed to the considerable substitution of Fe²⁺

for Al³⁺ in the octahedral layer which depresses the dissociation of unit layers of smectite (Egashira &

Ohtsubo, 1983). The low-swelling nature of smectite in Ariake clay was also confirmed by the swelling pressure tests

(Ohtsubo et al. 1985). Figure 9 shows the results of swelling pressure for Ariake clay sample (E-17)

with low-swelling smectite and Saga paddy soil (W-179-3) with high-swelling smectite. The swelling

pressure of Na-soil for sample W-179-3 was markedly affected by salt concentration, and the peak value

at 0.01 N was greater than that at 1.0 N salt concentration, whereas the swelling pressure of Ca-soil was

hardly influenced by salt concentration. For the sample E-17, salt concentration and cation valency both

Figure 9. Swelling pressure for Na and Ca saturated Ariake clay with high and low swelling smectite at salt different

concentrations (Ohtsubo et al. 1985).

Ariake 3 m Ariake 14 m

Figure 10. Scanning electron micrographs of Ariake clay at (a) 3 m depth, and (b) 14 m depth. Scale bars = 10 μ m

(Tanaka et al. 2001).

exerted little effects on the swelling pressure, suggesting little occurrence of intracrystalline swelling of

the smectite even after saturation with sodium. The low-swelling nature of smectite has been confirmed

in the smectite in Japanese marine clays other than Ariake clay (Egashira & Ohtsubo 1984).

2.3 Structure

Microstructural studies of the Ariake clay (3 and 16 m) (Tanaka et al. 2001) reveal the presence of

a well-developed flocculated structure combined with abundant fossil remains, mostly derived from

diatoms (Fig. 10). Aggregates are made up of agglomerated fine particles (mostly smectite) while the

coarse component is composed of quartz grains or fossil debris. The resulting pore space families are

intra-aggregate, interaggregate, and skeletal (Tanaka & Locat 1999, Shiwakoti et al. 1999). Since most

fossil remains have been found broken, very little intra-skeletal pore family is visible here.

3 POST-DEPOSITIONAL CHANGES

3.1 Chemistry of clay sediment

The chemistry of the clay sediment at Megurie, Okishin, Higashi-shiroishi, Ushiya, and Yamaashi is

shown in Figures 11 to 15. The clay sediment at Yamaashi has been subjected to salt leaching while those

Depth (m)	Na	K	Ca	Mg	Cl	pH	Fe	TOC (%)	CEC (cmol c / kg)	Organic matter (%)
0	7.9	6.6	8.1	8.0	8.0	7.8	8.2	7.9	8.0	7.9
10	7.9	6.6	8.1	8.0	8.0	7.8	8.2	7.9	8.0	7.9
20	7.9	6.6	8.1	8.0	8.0	7.8	8.2	7.9	8.0	7.9
30	7.9	6.6	8.1	8.0	8.0	7.8	8.2	7.9	8.0	7.9
40	7.9	6.6	8.1	8.0	8.0	7.8	8.2	7.9	8.0	7.9
0	7.9	6.6	8.1	8.0	8.0	7.8	8.2	7.9	8.0	7.9
10	7.9	6.6	8.1	8.0	8.0	7.8	8.2	7.9	8.0	7.9
20	7.9	6.6	8.1	8.0	8.0	7.8	8.2	7.9	8.0	7.9
30	7.9	6.6	8.1	8.0	8.0	7.8	8.2	7.9	8.0	7.9
40	7.9	6.6	8.1	8.0	8.0	7.8	8.2	7.9	8.0	7.9

Figure 11. Chemistry of clay profile at Megurie. 0 10 20 30 40 0 0.1 0.2 0.3 0.4 0.5 Ions in pore water (mol c /L) Depth (m) 7.9 6.6 8.1 8.0 8.0 8.0 7.8 8.2 7.9 8.0 7.9 7.9 7.8 8.0 7.4 7.4 4.0 3.9 3.9 4.0 5.2 pH 0.27 0.18 0.24 0.18 0.14 0.13 0.07 0.07 0.06 0.06 0.08 0.20 0.18 0.19 0.28 0.28 0.48 0.23 0.48 0.25 Fe 203 (%) 17.6 19.9 20.9 33.2 25.0 25.8 27.8 24.6 23.7 28.5 20.9 25.5 24.7 35.5 38.4 38.1 34.7 37.3 39.1 42.9 25.9 CEC (cmol c / kg) 1.47 1.03 1.24 1.56 1.47 0.99 1.12 1.16 1.18 1.15 1.01 1.85 1.27 1.33 1.40 1.45 1.49 1.45 1.84 2.22 Organic matter (%)

0.28	0.48	0.23	0.48	0.25	Fe	2	0	3	(%)	17.6	19.9	20.9	33.2
25.0	25.8	27.8	24.6	23.7	28.5	20.9	25.5	24.7	35.5	38.4			
38.1	34.7	37.3	39.1	42.9	25.9	CEC	(cmol	c	/	kg)	1.47	1.03	
1.24	1.56	1.47	0.99	1.12	1.16	1.18	1.15	1.01	1.85	1.27			
1.33	1.40	1.45	1.49	1.45	1.84	2.22	O	r	g	a	n	i	c
							m	a	t	t	e		
							r	(%)	Na	K	Ca	2	Mg
							2						Cl

Figure 12. Chemistry of clay profile at Okishin.

at Megurie, Okishin, Higashi-shiroishi and Ushiya have not. The Cl - concentration in the pore water

at Megurie (Fig. 11) and Okishin (Fig. 12) was almost constant from the base to a depth of 10 m and

increased markedly to the surface. This is roughly correspondent to the depositional salinities that would

be expected as the saline environment changed from brackish to marine through the JomonTransgression

during 10,000-6000 years B.P. (Fig. 2). The variation of the Na + concentration with depth was similar to

that of the Cl - concentration. The K + concentration was less than 0.03 mol c /L throughout the depth. The

mol c /L (SI unit) corresponds to the normality (N). The Ca 2+ and Mg 2+ concentration exhibited higher

values in a zone of 14-21 m than 0-14 m at Megurie, and in a zone of 25-44 m than 0-25 m at Okishin.

The pH of the clays varied from 3.9 to 9.3, with the lower values in the deeper zones.

For Higashi-shiroishi (Fig. 13) and Ushiya (Fig. 14), the Cl - concentration in the pore water increased

from the base to a maximum at 4-5 m, and decreased towards the surface. This corresponds roughly to 0.5 10 15 20 0.1 0.2 Ions in pore water (mol c /L) D e p t h (m) 3.9 4.8 5.4 5.3 7.3 5.4 6.0 6.9 6.4 7.5 7.2 7.0 5.9 7.1 4.5 4.0 3.8 3.7 pH 35.1 36.3 36.6 34.6 32.9 34.6 33.7 32.8 30.9 31.3 31.4 28.8 28.4 20.9 22.7 26.8 31.4 31.9 CEC (cmol c / kg) Na K Ca 2 Mg 2 Cl

Figure 13. Chemistry of clay profile at Higashi-shiroishi. 0.5 10 15 20 0.1 0.2 Cations in pore water (mol c /L)

Depth

(m)

0.0 0.1 0.2 0.3 0.4 Anions in pore water (mol c /L) SO₄
2- 3.0 5.2 7.3 7.5 4.7 7.6 7.6 5.0 7.5 7.5 6.3 7.5 7.1 6.4
3.2 6.6 4.7 5.9 4.3 pH 1.69 1.82 1.96 1.40 1.67 1.09 0.95
1.91 1.32 0.79 1.67 0.86 1.03 1.37 1.99 1.76 1.80 1.70 2.42
Fe²⁺ 0.3 (%) 2.97 2.99 2.83 2.48 2.24 2.09 2.18 2.67 2.40
2.09 2.61 2.50 2.30 2.28 2.78 2.33 2.58 2.97 2.59 Al³⁺ 2.0 3
(%) 21.2 22.4 18.7 12.5 15.7 15.1 19.3 18.5 16.5 9.5 8.8
11.2 8.4 5.5 8.3 3.9 5.9 8.3 SiO₂ (%) Na K Ca²⁺ Mg²⁺ Cl

Figure 14. Chemistry of clay profile at Ushiya. 0.5 1.0 1.5 0
0.02 0.04 0.06 0.08 0.1 Ions in pore water (mol c /L) Depth
(m) 3.8 3.7 3.8 4.6 5.0 4.1 4.0 3.9 4.1 4.9 4.7 4.0
3.8 3.8 3.7 3.8 pH 0.99 0.84 0.46 0.60 0.88 0.83 0.72 0.35
0.71 0.46 0.59 0.78 0.65 0.72 0.95 1.00 Fe²⁺ 0.3 (%) 35.8
32.6 17.5 27.0 32.7 31.7 28.3 39.6 28.1 25.8 27.9 29.0 33.6
29.3 32.6 31.3 CEC (cmol c / kg) 1.34 1.14 0.43 1.23 0.90
1.24 1.03 2.10 1.47 1.39 1.00 0.90 1.14 1.05 1.05 1.18 Organic
matter (%) Na K Ca²⁺ Mg²⁺ Cl

Figure 15. Chemistry of clay profile at Yamaashi. Table 2.
Composition of sea water. Ion mol c /L* % Na + 0.459 77.0 K
+ 0.010 1.7 Mg²⁺ 0.106 18.0 Ca²⁺ 0.020 3.4 Cl - 0.535
90.0 SO₄²⁻ 0.055 9.3 HCO₃⁻ 0.002 0.4 *corresponding to
normality (N)

changes in the depositional salinity from brackish to
marine through the Jomon Transgression during

10,000-6000 years B.P. and from marine to brackish due to
the lowering of the sea level after 3500

years ago (Fig. 2). The variation of the Na⁺ concentration
with depth was similar to that of the Cl⁻

concentration. The K⁺ concentration was less than 0.01 mol
c /L throughout the profile. The concentrations

of Mg²⁺ and Ca²⁺ in the pore water ranged from 0.01 to
0.1 mol c /L for Ushiya and less than 0.01 mol c /L

for Higashi-shiroishi. The concentration of the Ca²⁺ and
SO₄²⁻ at Ushiya were much higher than those

in the open sea (Table 2). The pH of the clays varied from 3.0 to 7.6, with the lower values in the

near-surface and the deeper zones at Ushiya and Higashi-shiroishi. The iron oxide content, ranging from

0.79 to 2.43%, showed higher values in the near-surface and the deeper zones. For the sediment at Yamaashi subjected to salt leaching the Cl⁻ concentration was below 0.03 mol c /L

at all depths, with lower concentrations at near-surface and deeper zones (Fig. 15). Cation concentrations

except K⁺ were greater than Cl⁻ concentrations, and the summation of cations exceeded Cl⁻ concen

trations sometimes with big differences. Some anions other than Cl⁻, such as SO₄²⁻, must be balancing

these excessive cations. The pH exhibited the values below 5 throughout the profile. The iron oxide and

organic matter content ranged from 0.35 to 1.0% and 1.03 to 1.47% respectively.

3.2 Salt removal

Leaching of salt in pore water is a major post depositional change in chemistry of clay sediment. For the

clay sediment at Yamaashi subjected to salt leaching (Fig. 15), the possible processes for salt removal

include diffusion of salt from high-concentration to low-concentration zones, leaching from above, lateral

leaching, and leaching from the base. The most probable explanation for the reduction in salinity below

0.03 mol c /L (as Cl⁻ concentration) is leaching from below in response to excess water pressures in the 2 3 4 5 6 7 8 0 1 2 3 pH F e 2 0 3 (%) R = 0.71 Significant at 1% level Ushiya 2 3 4 5 6 7 8 9 10 0 0.5 1 1.5 pH

F e

2 0

(%

) $R = 0.71$ Significant at 1% level Megurie Yamaashi

Figure 16. Correlations between iron oxide content and pH.

underlying coarse sediments (Torrance & Ohtsubo 1995). The landscape conditions are such that excess

water pressures may have existed in the coarse underlying sediments. The surrounding hills rise steeply

to elevation exceeding 200 m. The situation is not greatly different from that at Drammen, Norway, where

leaching from below is known to have occurred (Moum et al. 1971). Excess pumping up of groundwater

has been reported to be responsible for salt leaching in the Ariake clay sediment of the Kawazoe area

(Miura et al. 1996).

3.3 Weathering

The weathering is a natural consequence of the exposure of a soil material to a subaerial environment.

The greatest degree of weathering in the surface zone was confirmed in Norwegian and Canadian clay

(Moum et al. 1971). For Ariake clay the pH values of the profile were below 5.0 throughout the depth

at Yamaashi (Fig. 15), and at deeper and surface zones at Megurie, Okishin, Higashi-shiroishi, and

Ushiya (Figs 11 to 14). Such low pH values would be associated with the production of sulfuric acid

accompanied by the oxidation of pyrite in the clays (Lessard & Mitchell 1985, Adachi et al. 1992). The

presence of pyrites (1-2% in content) in the marine sediment of Ariake Bay area has been reported by

Kawasaki (1988) and Ohtsubo (1995). The oxidation of pyrite is

This reaction indicates that 2 mol sulfuric acid and 1 mol ferric hydroxide are ultimately produced

due to the oxidation of 1 mol pyrite, which leads to the reduction of pH of the clay sediment. This

is demonstrated by the higher iron oxide content and lower pH values at deeper and surface zones at

Megurie, Okishin, Higashi-shiroishi, and Ushiya, and throughout the depth at Yamaashi (Figs 11 to

15). These observations are summarized in Figure 16, where negative correlations exist between the

pH and iron oxide content for the combined data of Megurie and Yamaashi and for Ushiya. The SO_4^{2-}

concentrations in the pore water at Ushiya were higher than those in sea water (Table 2), and the SO_4^{2-}

concentration was correlated with the pH values of the clay (Fig. 17). These indicate that the extra SO_4^{2-}

was produced by reaction (1) through the exposure of a soil material to a subaerial environment. Thus, the

pH values and the relative amount of iron oxides and SO_4^{2-} concentration in the pore water in sediments

could be an index of the extent of weathering to which a clay material has been subjected.

The Ca^{2+} concentrations in the pore water were higher at deeper zones at Megurie (Fig. 11), Higashi

shiroishi (Fig. 13), and Ushiya (Fig. 14), which were above the Ca^{2+} concentration in the sea water

(Table 2). Excess Ca^{2+} would be a result of the dissolution of calcite due to the low pH caused by the

sulfuric acid production in reaction (1), which is described by

The Ca^{2+} concentration was correlated with sulfuric acid concentration for the profiles at Ushiya in

Figure 18. 2 3 4 5 6 7 8 0 0.1 0.2 0.3 0.4 pH SO_4^{2-} c o n

centration (mol c / L) R = 0.79 Significant at 1% level

Figure 17. Correlation of pH and sulfuric acid con

centration in pore water (Ohtsubo et al. 1995). 0 0.02 0.04 0.06 0.08 0.1 0 0.1 0.2 0.3 0.4 Ca²⁺ concentration (mol c /L) S O₄ 2 c o n c e n t r a t i o n (m o l c / L) R = 0.68 Significant at 1% level Figure 18. Correlation of calcium ion concentration and sulfuric acid concentration in pore water (Ohtsubo et al. 1995). 0 5 10 15 20 0 40 80 2 μ m 5 μ m 75 μ m

D

e p

t h

(m

) Particle size (%) 0 40 80 120160 w n w L w p Water content (%) γ_t : Wet density (kN/m³), s_u : Undrained shear strength (kPa) s_{ur} : Remolded shear strength (kPa) , s_t : Sensitivity, p_c : Consolidation yield stress (kPa) L i q u i d i t y i n d e x 0.98 1.31 1.21 1.78 1.51 1.31 1.34 1.63 1.42 1.45 1.27 1.21 1.38 1.45 1.51 1.22 1.18 0.93 1.45 0.94 γ_t 12.7 13.1 12.5 12.8 13.3 12.9 13.1 14.3 13.5 13.6 13.6 14.1 14.3 15.2 13.9 14.2 14.5 16.2 20 40 s_u s_{ur} 0.88 0.59 0.59 0.39 0.39 0.39 0.59 0.39 0.68 0.68 0.78 0.88 0.98 1.18 1.37 0.98 0.88 1.18 1.37 2.06 s_t 7 8 13 16 17 14 28 12 18 17 16 18 11 21 18 30 27 24 21 0 40 80 120160 p_c Effective over- burden pressure 0

Figure 19. Geotechnical properties of clay profile at Megurie.

4 BEHAVIORAL CONSEQUENCES OF POST-DEPOSITIONAL CHANGES

4.1 Physical properties

Figures 19 to 24 shows the geotechnical properties of the clay profile at Megurie, Okishin, Isahaya,

Higashi-shiroishi, Ushiya, and Yamaashi, respectively. The clay fraction (<2 μm) and silt fractions were

predominant throughout the depth at the sediment profiles except Okishin, while considerable sand

fraction was contained at the depths above 20 m at Okishin. The liquid limit was in a range of roughly 50

to 180% at Megurie, Ushiya, Isahaya, and Higashi-shiroishi, exhibiting a decrease with depth, while the

liquid limit was mostly in a range of 50 to 130% at Okishin and Yamaashi, exhibiting an increase with

depth at Okishin. The natural water content, having a similar variation to the liquid limit, gave higher

0 10

20

30

40

50 0 40 80 22 μ m 25 μ m 75 μ m

D

e p

t h

(m

) Particle size (%) 0 80 160 w n w L w p Water content (%)
 L i q u i d i t y i n d e x 1.18 1.85 1.13 1.11 1.37 1.05
 1.41 1.38 1.13 1.10 1.15 0.98 0.92 0.82 0.75 0.90 1.10 1.76
 0.42 0.58 0.69 γ t : Wet density (kN/m³) s u :
 Undrained shear strength (kPa) s ur : Remolded shear
 strength (kPa) s t : Sensitivity p c :Consolidation yield
 stress (kPa) γ t 15.3 15.8 15.4 14.8 14.5 14.9 15.0 14.8
 14.4 14.3 14.3 14.2 14.0 14.1 14.2 14.2 14.1 14.3 15.8 13.8
 11.8 16.1 0 40 80 s u s ur 1.47 1.51 1.52 1.31 1.41 1.42
 1.41 1.26 1.10 1.32 1.39 1.65 1.71 2.05 1.98 3.19 2.21 2.21
 2.34 5.61 s t 5 8 12 10 13 15 20 22 27 25 26 29 25 22 27
 16 25 23 20 10 0 80 160 240 p c Effective Overburden
 pressure

Figure 20. Geotechnical properties of clay profile at Okishin.

0

5

10

15

20

25 0 40 80 22 μm 25 μm 75 μm

D

e p

t h

(m

) Particle size (%) 0 40 80 120 w n w L w p Water content (%) 1.58 1.55 1.42 1.46 1.47 1.27 1.33 1.28 1.22 0.93 0.76
L i q u i d i t y i n d e x 13.0 12.7 13.2 13.2 13.3 13.5 13.7 14.8 14.1 14.5 14.3 15.0 γ t 0 20 40 s u s u /p 0.87
0.21 0.23 0.33 0.34 0.52 0.43 0.53 0.41 0.54 0.40 0.43 0.39
0.36 0.34 0.42 0.48 0.45 0.54 0.53 0.44 0.55 0.45 s u r 0.39
0.26 0.25 0.33 0.65 0.62 0.56 0.41 0.54 0.50 0.71 0.62 0.73
1.16 1.35 1.41 1.15 1.11 1.21 1.44 1.31 1.27 9.55 s t 14
7 11 16 10 19 20 38 25 39 23 31 26 17 15 18 28 29 34 30 29
39 5 0 40 80 120 160 p c γ t : Wet density (kN/m^3) s u
: Undrained shear strength (kPa) s u r : Remoulded shear
strength (kPa) s t : Sensitivity p c : consolidation
yield stress (kPa) Effective Overburden pressure

Figure 21. Geotechnical properties of clay profile at
Isahaya. 0 5 10 15 20 0 40 80 22 μm 25 μm 75 μm D e p t
h (m) Particle size (%) 0 80 160 w n w L w p Water
content (%) L i q u i d i t y i n d e x 1.21 1.38 1.06
0.91 0.94 1.21 1.84 1.08 1.17 1.26 1.48 1.36 2.41
1.41 1.13 1.26 1.38 1.19 γ t 13.4 12.0 13.6 13.9
13.9 14.3 13.7 14.1 14.3 14.4 14.5 15.1 15.3 16.4
16.1 15.3 14.9 15.4 0 20 40 s u γ t : Wet density
(kN/m^3) s u : Undrained shear strength (kPa) s u r :
Remolded shear strength (kPa) s t : Sensitivity p:
Effective overburden pressure (kPa) s u /p 0.58 0.61 1.15
0.40 0.60 0.58 0.43 0.48 0.42 0.52 0.47 0.49 0.40
0.33 0.45 0.26 0.54 0.45 s u r 0.46 0.25 0.47 0.36
0.46 0.64 0.39 0.50 0.42 0.50 0.51 0.73 0.75 1.02
1.20 0.84 1.36 1.64 s t 18 17 26 15 24 18 30 30 36 41
38 32 29 20 25 23 31 23

Figure 22. Geotechnical properties of clay profile at
Higashi-shiroishi. 0 5 10 15 20 0 40 80 22 μm 25 μm 75 μ

m

D

e p

t h

(m

) Particle size (%) 0 80 160 w n w L w p Water content (%)
 γ_t : Wet density (kN/m³) s_u : Undrained shear
strength (kPa) s_{ur} : Remoulded shear strength (kPa) s_t :
Sensitivity p_c : Consolidation yield stress (kPa) L i q u
i d i t y i n d e x 1.13 1.21 1.21 1.52 1.14 1.29 1.44 1.31
1.24 1.29 1.37 1.14 1.41 1.71 1.17 1.15 1.06 1.06 0.51 γ_t
12.8 12.8 12.1 13.8 12.9 13.3 13.3 13.1 13.2 13.3 13.4 13.5
14.1 14.3 14.0 14.3 14.5 14.3 26.3 0 20 40 s_u s_u/p 0.43
0.27 0.37 0.33 0.29 0.29 0.25 0.35 0.30 0.34 0.38 0.35 0.33
0.35 0.42 0.46 0.59 0.67 0.72 s_{ur} 0.50 0.22 0.16 0.39 0.57
0.65 0.44 0.44 0.52 0.39 0.63 0.68 1.01 1.47 0.59 1.17 1.43
2.94 s_t 27 46 117 39 21 25 26 36 30 41 38 35 24 26 42 24
23 14 0 40 80 p_c Effective Overburden pressure

Figure 23. Geotechnical properties of clay profile at Ushiya.

values than the liquid limit at most depths in all the sediment profiles. This was reflected in the liquidity

index greater than 1.0. The liquidity index was in a similar range (less than 1.5) at Megurie, Okishin,

Isahaya, Higashi-shiroishi, Ushiya while was in a higher range of 1.11 to 2.85 at Yamaashi. The liquidity

index greater than 1.0 could be explained in terms of flocculation, minimal consolidation and slow load

increase for clay sediments (section 4.4.2). 0 5 10 15 20 0
40 80 2 μ m 5 μ m 75 μ m D e p t h (m) Particle size
(%) 0 40 80 120 w n w L w p Water content (%) L i q u i d i
t y i n d e x 1.73 2.27 0.94 0.93 2.41 2.04 1.64 3.14 1.92
1.68 1.34 2.10 2.58 2.12 2.52 2.26 γ_t 13.6 14.8 15.5 13.7
14.6 14.6 14.5 14.5 15.6 16.1 15.5 14.9 14.7 15.1 15.2 15.4
0 20 40 s_u s_u/p 1.31 1.20 1.00 1.02 0.70 0.69 0.42 0.33
0.33 0.47 0.56 0.53 0.52 0.48 0.43 0.56 s_{ur} 0.32 0.30 0.27
0.41 0.18 0.18 0.19 0.25 0.27 0.61 0.19 0.21 0.09 0.06 0.03
0.03 s_t 20 37 46 48 92 109 73 51 54 38 176 187 366 636 895
1000

Figure 24. Geotechnical properties of clay profile at Yamaashi.

Figure 25. Histograms of specific gravity of clayey soil and silty soil; N = number of samples (Fujikawa & Takayama 1980).

The extents of the particle density, fraction $<5\ \mu\text{m}$, and consistency limits in Ariake clay were summa

rized by Fujikawa & Takayama (1980a) based on the data obtained by Yamaguchi et al. (1964). Figure 25

shows the histogram of specific gravity of the clay and silty soil. For the clay and silty soil the average

of the specific gravity was 2.629 and 2.684 and the confidence interval of the specific gravity was 2.56

to 2.70 and 2.61 to 2.76. The distribution of the frequencies for fraction $<5\ \mu\text{m}$ is shown in Figure 26.

The distribution pattern was similar in the northern and eastern coasts but was different in the western

coast from the others. These can be explained in terms of the sea water movement in Ariake Bay (Fig.1).

A counterclockwise water movement exists in the bay, which would sweep the finer suspended particles

delivered by the rivers on the east side towards the northern and western shorelines, where sedimentation

would occur. This would have brought about the higher clay fraction percentage in the western coast

than in the northern and eastern coasts. 0 20 40 60 80 0 10
20 30 40 Fraction ($<5\ \mu\text{m}$) (%) P e r c e n t a g e (%) :
Eastern region, N=89 : Northern region, N=109 : Western
region, N=197 N : Number of samples

Figure 26. Frequency distribution for $<5\ \mu\text{m}$ fractions (Fujikawa & Takayama 1980). 40 60 80 100 120 140 160 0 10
20 30 Liquid limit (%) P e r c e n t a g e (%) 20 30 40
50 60 70 0 10 20 30 Plastic limit (%) : Upperzone, N=207
: Deeper zone, N=119

Figure 27. Frequency distributions for consistency limits of clayey soil (Fujikawa & Takayama 1980). Figure 27 shows the distribution of the frequencies for the consistency limits of upper and deeper

zones in sediment. The distribution pattern was clearly different between the upper and deeper zones. For

the upper and deeper zones, the average liquid limit was 99.4 and 82.6%, respectively, and the average

plastic limit was 44.8 and 41.1%, respectively, where the difference in the averages was significant at

1% level for both liquid and plastic limits. Figure 28 shows the activity of Ariake clay. The ratio of plasticity index to clay fraction is termed the

activity, which was grouped into three classes: inactive (<0.75), normal ($0.75-1.25$) and active (>1.25),

based on the mineralogy and geological history of the clay deposits (Skempton 1953). The activity varied

in a wide range from 0.55 to 2.1, and these soil materials fall mostly into the normal and active clay

groups. The soil materials from the upper zone (0-10 m) gave higher activity than the deeper zone (below

10 m), which suggests that the liquid limit exhibits higher values in the upper zone than in the deeper

zone. Figure 29 shows the activity of clays from different parts of the world (Tanaka et al. 2001). Ariake

clay exhibits the highest values among those although there is a wide scatter in the activity. The plasticity

chart for the three clays in Figure 29 is shown in Figure 30 (Tanaka et al. 2001), where w_L and I_p are

plotted on the horizontal and the vertical axes, respectively. The data for Ariake clay fall along the A line.

4.2 Factors affecting consistency limits

The liquid and plastic limits of marine clay are governed by the nature of the solid component of soil

materials and their chemical properties. The nature of the solid component includes grain size, clay

Figure 28. Activity of Ariake clay.

Figure 29. Comparison of the activity of clays with different origin (Tanaka et al. 2001).

mineral composition, organic matter, and diatom content etc., and the chemical properties include pore

water salinity, exchangeable cation, pH, and oxides of iron and aluminum, that can be altered by a

change in the chemical environment of the clay sediment. Of these controlling factors of the liquid and

plastic limits, we will consider the clay mineral composition and the pore water salinity since they vary

to a great extent in the clay profile (Figs 11-15). For clay mineral composition, Figure 31 shows the

relationship between the activity and smectite content in bulk soil for the Ushiya profile of Figure 23.

There is a significant positive correlation between the two parameters, suggesting that the samples in

the upper zone with higher smectite content gave higher liquid limit than the samples in the deeper zone

with lower smectite content. Higher liquid and plastic limits for the clay samples with higher smectite

content are ascribed to higher water sorption capacity due to greater specific surface area (De Kimpe

et al. 1979, Locat et al. 1984). Similarly higher smectite percentage in the upper zones contributes to the

Figure 30. Plasticity index versus liquid limit relationship for clays with different origin (Tanaka et al. 2001).
20 30 40 50 60 0.8 1.2 1.6 2 Smectite content (%) A
c t i v i t y Upper zone (0-14 m) Deeperzone (14-18 m) R $\square$
0.850 Significant at 1% level

Figure 31. Correlation between activity and smectite content in bulk soil for Ushiya clay profile (Ariake).

higher liquid limits in Megurie and Yamaashi clay profiles of Figures 19 and 24. Significant correlations

between the consistency limits and smectite content were also found for clay samples with different

geological origin (Fig. 32) (Ohtsubo et al. 2002). For the effects of pore water salinity on the consistency limits, the higher pore water salinity in the

upper zone of the clay profiles would enhance the liquid limit (Figs 11-15 and Figs 19-24). To assess

how the change in pore water salinity affects the liquid limit, the liquid limits were measured on the

clay samples prepared artificially at different salt concentrations in the pore water. Figure 33 shows the

liquid limit thus determined on the samples obtained from different depths at Megurie in Figure 19. For

all the samples, their liquid limit decreased dramatically with decreasing chlorine ion concentration in

a range less than about 0.2 mol/L. These results suggest that the higher liquid limits in the upper zones

of the clay profiles at the site except Okishin (Figs 19, 21-24) are caused by higher pore water salinity. The effects of reduction in pore water salinity on the liquid limit were observed at Yamaashi where

salt leaching has occurred in the clay sediment (Fig.24). Figure 34 shows reductions in the activity due

to salt leaching in the clay sediment using the data for the non-salt-leached sediment at Higashi-shiroishi

(Fig.22) and the salt leached clay sediments at Yamaashi (Fig.24). The activities at the same depths for

the clay sediments were connected with straight lines.
Reduction in the activity due to the increase in
0 10 20 30
40 0 50 100 150 200 250 Smectite content in bulk soil (%) L
i q u i d a n d p l a s t i c l i m i t s (%) LL PL
Hachirogata Ariake Singapore Pusan y = 60.76e - 0.031x y =
1.12x - 22.2 r = 0.948** **Significant at 1% level r =
0.966**

Figure 32. Correlation between consistency limits and smectite content in bulk soil for clays with different origin

(Ohtsubo et al. 2002). 0 0.2 0.4 0.6 80 100 120 140 L i q u i d l i m i t (%) Depth of samples 1.5 m 2.5 m 5.3 m Chloride ion concentration (mol/L)

Figure 33. Variation in liquid limit with pore water salinity for samples from different depth (Ohtsubo et al. 1996).

pore water salinity indicates that the plasticity index is decreased by reductions in pore water salinity

followed by salt leaching.

4.3 Shear strength

4.3.1 The ratio of the undrained strength, S_u , to the consolidation pressure, p

The undrained shear strength, S_u , and S_u/p values of the clay sediment are presented in Figures 19 to 24.

The S_u/p value is represented as the ratio of the undrained shear strength, S_u , to the effective overburden

pressure, p , for the sediment. A clear change in the shear strength was observed at a certain depth of all

clay profiles. The shear strength showed linear increases with depth for both upper and deeper zones,

with a greater increasing ratio for the deeper zone. This is demonstrated by the magnitude of the S_u/p

values, giving greater S_u/p values for the deeper zones. Two explanations for the greater S_u/p values for

the deeper zones are possible. One is the difference in the salinity environment when the sedimentation

occurred: low salinity in the deeper zone at the time of deposition would have produced oriented structure

(van Olphen 1977), giving the higher shear strength compared with the flocculated structure developed

Figure 34. Reduction in the activity of clays by salt leaching (Ohtsubo et al. 1999). 0.5 1.0 1.5 2.0 0.2 0.3 0.4 0.5 0.6 0.7 Iron oxide (%) 0-3 m 3-14 m 14-19 m Ushiya R 0.51 Significant at 5% level S u / p 0.2 0.4 0.6 0.8 1.0 1.2 0.2 0.3 0.4 0.5 0.6 0.7 Iron oxide (%)

S u

/ p Megurie R 0.47 Significant at 5% level

Figure 35. S u /p value as a function of iron oxide content (Ohtsubo et al. 1995).

in the upper zones under high salinity environment (Bjerrum & Rosenqvist 1956). The other is the post

depositional changes in the chemistry due to weathering accompanied by the exposure of the sediment

surface to the atmosphere. The iron oxide produced by pyrite oxidation after sedimentation by reaction

(1) contributes as a cementing agent to enhance the shear strength in the deeper zone. The higher S u /p

value in the surface zone at Isahaya (Fig. 21), Higashi-shiroishi (Fig. 22), and Ushiya (Fig. 23) would

also have been brought about by the iron oxide produced through pyrite oxidation in the surface zone.

The surface zone would have been exposed to the atmosphere when the sea water level lowered below

the present level during 3000-1000 B.P (Fig. 2). The correlations of S u /p value and iron oxide content for Megurie and Ushiya are illustrated in

Figure 35, where S u /p values increased with increasing iron oxide content and the surface and deeper

zones with higher iron oxide contents gave higher S u /p values. The results for Megurie and Ushiya were

shown separately since the method to determine the iron oxide content was different; the iron oxides in

the soil were extracted with dithionite and citrate for Megurie (Ohtsubo et al. 1983) and with Tiron for

Ushiya (Ohtsubo et al. 1995). The contribution of cementing agents including iron oxide to the shear

strength has been reported in several studies (Newill, D. 1961, Kenny et al. 1967, Townsend et al. 1971,

Loiselle et al. 1971, McKyes et al. 1974, Yong et al. 1979, Torrance 1990). Figure 36 shows the change

in the Bingham yield stress for illite by the addition of iron oxide determined for the suspension of

illite-iron oxide mixtures using a rotation viscometer. The specimen was prepared by mixing the illite

and iron oxide at pH 9.5, followed by adjusting to different pH values. Addition of iron oxides of 2 and

Figure 36. Variation in Bingham yield stress with pH for illite - iron oxide complexes of initial pH 3.0 with different

iron oxide contents (Ohtsubo et al. 1991).

Figure 37. Relationship between S_u/p and plasticity index.

5% to the illite markedly increased the yield stress over a whole pH range. The enhancement of the shear

strength by iron oxide can be attributed to the increase of the interparticle attraction by association of

iron oxide particles with negatively charges clay surfaces (Yong & Ohtsubo. 1987, Ohtsubo et al. 1991).

A correlation of increasing S_u/p ratio with increasing plasticity index has been indicated for Norwegian

clays by Skempton & Henkel (1953), Bjerrum & Rosenqvist (1956), and Bjerrum (1967). Figure 37

shows the S_u/p ratio plotted against plasticity index, I_p , for Ariake clay samples along with the Skempton

Bjerrum correlation. It has been suggested that for clays deposited under marine conditions and are found

in the field with very loose structures, the Skempton-Bjerrum correlation represents a lower limit for the

S_u/p ratio for any given plasticity index. If a clay is deposited under different conditions, for example in

fresh water, a much higher S_u/p ratio is obtained (Bjerrum & Simons 1960). For Ariake clay no correlation

was observed between the two parameters and the S_u/p ratio was much higher than the Skempton-Bjerrum

correlation for a broad range of plasticity index. This is attributed to the difference in the composition

and geological history of Ariake clay sediment from those of Norwegian clay sediments. Ariake clay is

an alluvial deposit formed in a marine or brackish environment and consists of smectite as a principal Ground surface $\alpha \alpha \ 90^\circ \alpha \ 60^\circ \alpha \ 45^\circ \alpha \ 30^\circ \alpha \ 0^\circ$

Figure 38. Schematic diagram for the specimens for tests trimmed in different directions (Higashi & Takayama

1985).

clay mineral, while Norwegian clays are glacial deposits with finely ground rock flour and consists

predominantly of mica, chlorite, and primary minerals. Enhanced shear strength, S_u , in Ariake clay

would be a result of remarkable thixotropic hardening because of the presence of smectite (as described

in thixotropic processes later). Tanaka (2002) demonstrated an increase in the undrained shear strength of marine clay by addition of

diatoms and suggested that a high S_u/p ratio for Japanese clays including Ariake clay can be attributed to

diatoms since they are present in great amount in Japanese clays but are scarcely present in clays other

than Japanese ones. Ariake clay sediment contains as much as 22% silica which is a main component of

diatoms (Fig.14). Thus, the higher S_u/p ratio for Ariake clay than for Norwegian clay in Figure 37 would

be a combined effect of predominance of smectite and presence of diatoms in Ariake clay.

4.3.2 Anisotropy of shear strength

Several studies have been made on the anisotropy of undrained strength of soils (Hansen & Gibson 1949,

Lo 1965, Duncan & Seed 1966, Davis & Christian 1971). Some attempts have been made to study the

anisotropy of undrained strength of Ariake clay (Onitsuka et al. 1976, Muraoka 1979, 1980, Higashi &

Takayama 1985, Fujikawa et al. 1987). Higashi & Takayama (1985) conducted undrained shear strength

tests on undisturbed Ariake clay samples collected from different depths. Specimens for the tests were

trimmed in different directions from 0 to 90 degrees, which are presented schematically in Figure 38.

The shear tests were performed on each of these specimens for the two shear directions termed as active

and passive shearing, which is shown schematically in Figure 39 for the case of $\alpha = 45^\circ$. The A direction

represents active shearing, which is shearing toward the same direction as the shearing forces act during

one dimensional consolidation, while the P direction represents passive shearing, which is shearing

toward the opposite direction to active shearing. Figure 40 shows the shear stress versus displacement

curves obtained by active and passive shearing for specimens trimmed in different directions. Marked

differences in the shape of shear stress curves and peak strength values were observed among the

specimens. The active shearing gave higher shear strength than the passive shearing, and the active

shearing for the sample of $\alpha = 45^\circ$ gave the highest shear strength. To view how the anisotropy in shear

strength is varies with the angle (α) between horizontal plane and the direction of trimming (Fig. 38),

the variation in undrained shear strength with the angle, α , is shown in Figure 41 for the samples from

different depth. Positive and negative signs for α indicate active and passive shearing respectively. In all

the samples, the maximum shear strength appeared in a range of 45 to 60 ° for active shearing while the

minimum shear strength appeared in a range of -30 ° to -45 ° for passive shearing. A clear difference

in the extent of anisotropy was observed between the samples from 1 to 12 m and from 13 to 17 m; the

samples in the deeper zone indicated greater anisotropy in strength compared with the upper zone. The anisotropy of clays is intimately connected with their structure, which depends on the environmen

tal conditions during which the soil is deposited as well as the stress changes subsequent to deposition.

A review of the concept of structure has been given by Rosenqvist (1959), who demonstrated that clays

laid down in salt water acquire an open card house structure with the particles randomly oriented. In a 45 ° A 45 ° A 45 ° 45 ° Y 45 ° P 45 ° F View from F direction Ground surface $\alpha = 45^\circ$

Figure 39. Shear direction for the specimen trimmed in a direction of $\alpha = 45^\circ$. (Higashi & Takayama 1985).

Figure 40. Shear stress versus displacement curves for sample B (Higashi & Takayama 1985).

fresh water deposit, the structure is somewhat dispersed and a certain degree of parallelism is achieved

between the clay particles. It is conceivable, therefore, that in the former case the clay is more or less

isotropic in a macroscopic scale, while in the latter case, the clay will possess some inherent anisotropy.

As described previously, the upper Ariake clay sediments

above 12 m were deposited under a saline envi

ronment while the deeper sediments below 12 m were deposited under a brackish environment. Hence,

the higher extent of anisotropy in the deeper sediments than in the upper ones (Fig. 41) can be attributed

to the difference in the depositional environment in terms of pore water salinity. It is known that con

solidation under deviatoric pressures tends to align the clay particles. Under heavy overburden pressure,

therefore, a clay that is initially isotropic may become anisotropic. The higher extent of anisotropy in

deeper sediments might be partly due to such heavy overburden pressure.

Figure 42 shows the typical test results in active and passive shear directions for the specimen

of $\alpha = 45^\circ$ conducted on undisturbed samples (C-5 and C-9) obtained from the depths of 5 and

9 m, where a constant volume-direct shear test was used. The captions of shear directions corre

spond to those indicated in Figure 39. The magnitude of the shear strength, S_u , was in the order of

$S_{u45A} > S_{u45(45A)} > S_{u45Y} > S_{u45(45P)} > S_{u45P}$. Thus, due to changes in shear direction from active to pas

sive shearing, the shear strength, S_u , decreased and the peak in the strength versus displacement curves

became less distinct. Similar tendencies were also observed in the strength versus displacement curves for

the cases of $\alpha = 30^\circ$ and 60° (Higashi & Takayama 1985). To view the extent of anisotropy in strength for

active and passive shearing, the strength ratios, S_{u45A}/S_{u0} and S_{u45P}/S_{u0} , were introduced. The S_{u45A} , S_{u45P}

and S_{u0} indicate the active and passive shear strength for $\alpha = 45^\circ$, and the shear strength for $\alpha = 0^\circ$,

Figure 41. Variation of undrained shear strength, S_u , with trimming direction, α , for specimen; the number of the figures represent the depth (m) of the samples (Higashi & Takayama 1985).

Figure 42. Shear stress versus displacement curves for different shear direction determined on the specimen trimmed

in a direction of $\alpha = 45^\circ$ (Higashi & Takayama 1985).

respectively. The strength ratios for Ariake sample A with depth are shown in Figure 43. The values of

S_{u45A}/S_{u0} and S_{u45P}/S_{u0} were in a range of 1.1 to 1.3 and 0.8 to 0.9, respectively.

4.3.3 Remolded shear strength

Ariake clay is characterized by a wide range of sensitivity, exhibiting values of usually 7 to 50 and

occasionally up to 1000 (Figs 19-24). The higher sensitivity of the sediment is associated with higher

Figure 43. Changes in strength ratio, S_{u45A}/S_{u0} , S_{u45P}/S_{u0} , $(S_{u45A} - S_{u45P})/S_{u0}$, with depth for samples A and B

(Higashi & Takayama 1985).

Table 3. Physical properties of the samples. w_L w_L w_p $\tau_r(w_L)$ $\tau_r(w_L)$ Sample (%) (%) (%) (kPa) (kPa)

Saga City Asb 78.8 72.5 32.6 0.94 1.50

Ariake kantaku Ari 85.0 82.0 47.6 1.50 1.90

Shiraishi Asi 99.7 101.0 46.5 2.30 1.83

Bentonite (1) Bmy 225.0 168.0 30.9 0.51 1.72

Bentonite (2) Bho 271.0 222.0 37.9 0.76 1.65

liquidity index and lower remolded shear strength; hence a correlation between the liquidity index and the

remolded shear strength would be expected. Table 3 presents the physical properties of the clay samples

B are bentonite). The $\tau_r(w_{LB})$ and $\tau_r(w_{LF})$ represents shear strength at the water content equivalent to

the liquid limit determined by the Casagrade and fall cone methods, respectively. Figure 44 shows the

remolded shear strength as a function of the water content (Takayama 1971). Using these data and the

liquid and plastic limits in Table 3, the remolded shear strength versus liquidity index relationships were

plotted using the liquid limit obtained by the Casagrade cup and fall cone methods (Fig. 45). Though

the remolded shear strength decreased linearly with increasing liquidity index for the liquid limit by

both methods, more scatter exists in the correlation for the w_{LB} than for the w_{LF} . This can be explained

in terms of the difference in the remolded shear strengths, $\tau_r(w_{LB})$ and $\tau_r(w_{LF})$: the bentonites with

extremely high liquid limit exhibit lower remolded shear strength than the Ariake clay with low liquid

limit. Therefore the liquid limit by the fall cone method is desirable to be used for the liquidity index

in establishing the remolded strength versus liquidity index relationship. The correlation between the

liquidity index and the remolded strength in Figure 45 agrees with the results obtained by Skempton &

Northy (1952), and Locat & Lefebvre (1986).

4.4 Sensitivity

Sensitivity is defined as the ratio between the undrained shear strength of a soil material when it is

undisturbed and when it is completely remolded. A scale of sensitivities was proposed by Skempton &

Figure 44. Relationship between remolded shear strength and water content (Takayama. 1971).

Figure 45. Relationships between remolded shear strength and liquidity index (Takayama 1971). Table 4. Sensitivity scale (Skempton and Northy, 1952). Sensitivity Designation 1 Insensitive clays 1-2 Clays of low sensitivity 2-4 Clays of medium sensitivity 4-8 Sensitive clays >8 Extra sensitive clays >16 Quick clays

Northy (1952) (Table 4) and subsequently modified by Rosenqvist (1953) (Table 5). Torrance (1983)

proposed the sensitivity scale for clays currently used in Norway (Norsk Geoteknisk Forening 1974) to

be desirable in the light of historical changes of quick clay definition. This scale designates the sensitivity

as being low if less than 8, medium-high if 8-30, and high if over 30 (Table 6). The scale for quick clay

requires that the sensitivity must exceed 30 and the remolded shear strength must be less than 0.5 kPa.

This means that the remolded material must behave as a viscous rather than as a plastic solid. The sensitivity and remolded shear strength of the Ariake clay sediments is presented in Figures 19

to 24. The sensitivity of the sediment at Megurie, Okishin, Isahaya, Higashi-shiroishi, Ushiya ranged

from 7 to 117, which is mostly designated as being medium-high or high sensitivity since the remolded Table 5. Sensitivity scale (Rosenqvist, 1953). Sensitivity Designation 1 Insensitive clays 1-2 Slightly sensitive clays 2-4 Medium sensitive clays 4-8 Very sensitive clays 8-16 Slightly quick clays 16-32 Medium quick clays 32-64 Very quick clays >64 Extra quick clays Table 6. Sensitivity scale (Norsk Geoteknisk Forening, 1974). Sensitivity Designation <8 Low sensitivity 8-30 Medium high sensitivity >30 High sensitivity* *To be called 'quick clay' the remolded soil must be a fluid i.e. it has a remolded shear strength <0.5 kPa

shear strength is mostly greater than 0.5 kPa. The sensitivity of the sediment at Yamaashi ranged from

18 to 1000, which is mostly designated as being of quick clay since the remolded shear strength is less

than 0.5 kPa.

Torrance (1983) proposed a general model for the factors producing high sensitivity in clayey sediments

or quick clay development (Fig. 46). The material property requirements are that the sediment

must be dominated by minerals of low activity (i.e. non-swelling minerals), and that it must have a

high-water-content flocculated structure, such as develops upon sedimentation in a marine environment.

Finally post-depositional factors can lead to a sufficiently low remolded strength to behave as a liquid

after disturbance and thereby qualify as quick clay. For marine clays that meet the materials requirement,

the only essential post-depositional factor is leaching to decrease the pore-water salinity, but other factors

such as cementation, and thixotropic processes which increase the undisturbed strength, or dispersants,

which can further lower the remolded strength below that produced by leaching alone, can enhance the

sensitivity developed but are not generally required. It is interesting to compare the Ariake clay materials

with this model.

It is assumed that the Ariake clay is flocculated as this is the normal state for clays sedimented in

marine or brackish environments. Likewise, there is no reason to suspect that cementation (Fig. 35)

or slow load increase would have a different effect in other deposits that have been reported on in the

literature. The other factors influencing the development of sensitive and quick clay will be discussed

in the following order: material properties, 'thixotropic' processes in post-depositional factors which

contribute to increasing the undisturbed strength, and

post-depositional factors which contribute to

decreasing the remolded shear strength.

4.4.1 Low swelling clay minerals dominate

The dominant minerals in the clay-sized fraction of quick clays of Scandinavia and Canada typically

include quartz, feldspar, amphibole, hydrous mica and chlorite, with never more than trace amounts

of swelling clay minerals (Brydon & Patry 1961, Bentley & Smalley 1979). Reduction in pore-water

salinity for such clays with low active minerals decreases the liquid limit and remolded shear strength,

leading to an increase of sensitivity. The presence of swelling clay minerals normally inhibits or prevents

the development of high sensitivities (Torrance 1975). Ariake clay with smectite-dominated mineralogy,

however, responded to salinity and ion saturation in a manner similar to the low-activity minerals, illite

and chlorite, rather than in the manner normally associated with smectites (Figs 7 and 8). This unusual

behavior is due to the low-swelling and low-activity character discussed earlier (Egashira & Ohtsubo FACTORS PRODUCING A HIGH UNDISTURBED STRENGTH DEPOSITIONAL Flocculation 1,2 - salinity - divalent cation adsorption - high suspension concentration Cementation bonds - rapidly developed - slowly developed Slow load increase - time for cementation - "thixotropic" processes FACTORS PRODUCING A LOW REMOLDED STRENGTH DEPOSITIONAL Material properties - low-activity mineral dominated 1,2 Minimal consolidation Leaching 1 - decrease in w_L - decrease in w_L Dispersants 2 - decrease in w_L - decrease in w_L Essential in marine clays 2 Essential in fresh water clays POST-DEPOSITIONAL POST-DEPOSITIONAL

Figure 46. General model for quick clay development (Torrance, 1983).

1981). Thus Ariake clay material conforms, due to the dominance of low activity minerals, to the general

model in Figure 46.

4.4.2 Flocculation, slow load increase and minimal consolidation

Ariake clay sediments consist of fine particles with pyroclastic and granitic origin, which were trans

ported by fresh water streams and rivers to a marine or brackish environment. A flocculated structure

develops at the time of deposition as a consequence of one or more of: high salinity, dominance of diva

lent cations on cation adsorption sites. The electron micrographs for Ariake clay sediment in Figure 10

and the electron micrographs observed by Onitsuka et al. (1976) confirm the presence of flocculation.

The effect of the flocculated structure is to provide a framework with an inherent strength which, after

it has formed and stabilized, retains the sediment at a higher void ratio. The amount of consolida

tion is minimized by the flocculated structure, but slow load increase or the presence of only a low

overburden, or the rapid development of cementation after sedimentation will augment the sensitivity

compared with more heavily loaded or uncemented material. The final void ratio will thus be higher

than for more rapid load increase (Torrance 1987). Figure 47 shows the sensitivity versus liquidity

index relationship obtained from the data for the sediments in Figures 19 to 24. The sensitivity of 8

to 50 and the liquidity index above 1.0 for non-leached clays can be explained in terms of the slow

load increase and minimal consolidation mentioned above which occurred for a long period of 12,500

years.

4.4.3 Thixotropic processes

The term 'thixotropy' was introduced by Peterfi (1927) and used by Freundlich (1935) to describe

the well-known phenomenon of isothermal, reversible gel-sol transformation in colloidal suspensions.

Burgers & Scott-Blair (1949) defined thixotropy as a process of softening caused by remolding followed

by time-dependent return to the original harder state. For the cause of thixotropy, an hypothesis based

on initial non-equilibrium of interparticle forces after remolding, and the effects of this non-equilibrium

on subsequent structure changes within the soil has been suggested as one possible explanation of the

phenomenon by Mitchell & Houston (1960). Thixotropic effects in remolded natural clays have been studied by Moretto (1948), Skempton &

Northy (1952), and Takayama (1974). These investigations were aimed at determining the extent to

Figure 47. Sensitivity versus liquidity index for Ariake clay (Takayama 1995).

Figure 48. Shear strength versus rotation angle curves determined by vane shear tests for Ariake-kantaku samples

rested for different periods (Takayama 1976).

which thixotropic hardening could contribute to the sensitivity of clays. Slow load increase will allow

any thixotropic processes, which will cause an increase in the undisturbed strength, time to act (Mitchell &

Houston 1960). Takayama (1974) conducted vane shear tests to measure the strength increases with time

for Ariake clays maintained at various water contents. Figure 48 shows the results of vane shear tests

conducted on clay samples allowed to rest for different periods under a constant water content equal to

the liquid limit. These results clearly indicate that

Ariake clay acquires considerable strength as a result

of thixotropy. Similar tests were conducted on the samples prepared at different water contents, and the

thixotropic strength acquired is shown in Figure 49 for the respective samples as a function of aging time.

The water content of the samples is represented by the liquidity index. The strength increased linearly

with time for all the samples, exhibiting higher rate of increase in the strength for higher liquidity index.

The thixotropic strength ratio (R_t) was introduced by Seed & Chan (1957) to compare the magnitude of

thixotropic strength increase.

Thixotropic strength ratio = (Strength of specimen tested after time t) / Strength of specimen tested immediately after remolding)

Figure 49. Changes in shear strength for the samples with different water contents rested for different periods

(Takayama 1974).

Figure 50. Relationship between thixotropic strength ratio and resting time after remolding (Takayama 1974).

The thixotropic strength ratio after time t was calculated for the respective strength line, and the strength

ratio versus time relationship is shown in Figure 50. These correlations were formulated by the following

equation.

where R_t is the thixotropic strength ratio, t is time after remolding, t_1 is 1 day, and the values of a and b

are constants. The values of a and b were obtained for each thixotropic strength line, and were plotted as a

function of the liquidity index in Figure 51. The values of both a and b increased with increasing liquidity

index, indicating the higher liquidity index of the clay samples induces higher thixotropic strength. The

thixotropic strength ratio in a given period was estimated using equation (3), and the values of a and b

at different liquidity index in Figure 51 was compared with the sensitivity of natural Ariake clay. In comparing the sensitivity of clay with that acquired due to thixotropy, it is necessary to consider the

age of the undisturbed soil and the probable increase in thixotropic strength throughout the entire period

Figure 51. Relationships between constants, a and b, and liquidity index (Takayama 1974).

Figure 52. Comparisons of the sensitivity for different periods acquired by thixotropy (curves) with the sensitivity

for natural Ariake clay sediments (Takayama 1976).

since its deposition. Figure 52 shows the comparisons of the sensitivity for different periods acquired

by thixotropy (curves) with the sensitivity for natural Ariake clay sediments. The sensitivity curves for

different periods were obtained by incorporating a and b values (Fig. 51) corresponding to the liquidity

index of natural Ariake clay sediments into equation (3). It appears that thixotropy can account for a

great part of the original sensitivity of natural clay.

4.4.4 Leaching

The sediment at Yamaashi with the sensitivity of 18 to 1000 was designated as being quick clay (Fig. 24).

Figure 53 shows the undisturbed and remolded strength versus water content relationship for quick clay

(Yamaashi) and non-quick clay (Higashi-shiroishi). The undisturbed strength decreased with increasing

water content for both Higashi-shiroishi and Yamaashi. The remolded strength exhibited a decrease with

increasing water content for Higashi-shiroishi while

exhibited wide scattering in a smaller strength range

for Yamaashi. Taking into account that a decrease in the remolded strength is caused by a reduction in pore

water salinity, the remolded shear strength versus pore water salinity relationship is shown in Figure 54,

where a positive correlation was found between the two parameters. Based on the data of undisturbed

and remolded shear strength of clays, the sensitivity was correlated with salt concentration in pore water

in Figure 55. The data from laboratory tests conducted on undisturbed clay samples is also plotted. These

results indicate that salt removal by leaching is essential for the development of quick clay. 50 100 150 200 10 20 1 10 10 2

U n

d r

a i

n e

d

s h

e a

r s

t r e

n g

t h

(k

P a

) Undisturbed strength Higashi-shiroishi Yamaashi Remolded strength Higashi-shiroishi Yamaashi Water content (%)

Figure 53. Relationships between undisturbed and

remolded shear strength and water content for leached and

non-leached clays (Ohtsubo et al. 1982). $0.1 \leq e \leq 1.0$ $0.01 \leq \sigma'_v \leq 10$ kPa
Remolded shear strength (kPa) Salt concentration in pore water (g/L) Higashi-shirioshi Yamaashi
Figure 54. Correlation between remolded shear strength and pore water salinity for combined data of leached and non-leached clays (Ohtsubo et al. 1982). Salt leaching tests were conducted on undisturbed Ariake clay samples (Ariake kantaku) to demon

strate that salt removal from clays by leaching is essential to quick clay development. The undisturbed clay

samples were placed in consolidation rings with 6 cm in diameter and 2 cm in height, and a load equiv

alent to the overburden pressure was applied on the samples. Deionized water in different amount was

percolated through the clay samples, and the undisturbed and remolded strength of the samples were deter

mined by vane shear tests. The test results are presented in Table 7. The repulsive forces between particles

increase as pore water salinity decreases. This increase in repulsive forces decreased the liquid limit and

the remolded strength of the samples while the flocculated structure in a undisturbed state prevents con

solidation, leading to the natural water content and undisturbed strength being unchanged. The net result

of reductions in pore water salinity was to increase the liquidity index and sensitivity of the clay samples.

4.5 Compressibility

4.5.1 Compression index

Compression indices are frequently determined quantities in soil investigation programs, and this makes

them ideally suited for statistical treatment. Skempton (1944) proposed the following relationship

between compressibility index and liquid limit for remolded clays:

Terzaghi & Peck (1967) proposed a similar relationship for undisturbed clays with low and medium

sensitivity.

Since then, numerous single and multiple correlations between compression index and basic physi

cal properties such as natural water content, initial void ratio, and liquid limit have been proposed

(Balasubramanian & Brenner 1981), with the purpose of predicting compression index from simple 0.01 0.1 1 10 10 100 1000 S e n s i t i v i t y Salt concentration in pore water (g/L) Natural sediment Unleached Leached Data for leaching test A-4 A-12

Figure 55. Sensitivity versus pore water salinity for leached and non-leached clays.

Table 7. Changes in the chemical and geotechnical properties of undisturbed Ariake-kantaku samples by salt leaching

(Ohtsubo et al. 1982). Over Clay Pore Consistency Shear burden fraction water limits strength pressure Depth (<2 μ m) V salinity w (%) (kPa) S t

Sample (kPa) (m) (%) (mL) (g/L) (%) w L w p A c L I S S r (S/S r)

A-4 12.6 4.2-4.3 52.8 0 5.27 160 120 63 1.08 1.70 7.43 0.33 23 20 3.98 153 117 59 1.10 1.62 9.22 0.29 32 40 2.75 172 110 49 1.15 2.03 9.22 0.22 42 60 1.23 169 103 56 0.89 2.43 8.97 0.05 179 80 0.05 161 95 56 0.75 2.66 7.18 0.01 718

A-3 32.3 8.5-8.6 45 0 5.38 126 105 42 1.39 1.34 9.64 0.44 22 67 3.45 125 89 41 1.06 1.76 8.85 0.25 35 164 0.23 130 78 45 0.73 2.58 9.47 0.01 947

A-12 50 12.4-12.5 26 0 3.1 68 46 34 0.46 2.77 13.52 0.59 23 30 0.94 62 43 27 0.61 2.18 18.10 0.36 50 60 0.58 58 39 30 0.36 3.02 12.55 0.10 126 80 0.23 58 37 29 0.30 3.69 14.60 0.02 730

V : amount of percolated water, w: water content, A c : activity, LI: liquidity index, St: sensitivity

laboratory test which are less costly than oedometer tests. For undisturbed Ariake clay, several equations

have been proposed.

Uchida & Matsumoto (1959) $C_c = 0.29(w_L - 50)$

Yamaguchi et al. (1964) $C_c = 0.012(w_L - 4)$ $C_c = 0.36(e_0 - 10)$

Park & Koumoto (1998) $C_c = 0.016(w_0 - w_p) + 0.251$

Higashi et al. (2002) $C_c = 0.343e_0^{1.328}$

where e_0 and w_0 denote the natural void ratio and natural water content of the clay sediments, respectively.

A correlation between compression index and liquid limit for remolded Ariake clay proposed by

Fujikawa & Takayama (1980b) is given as

Figure 56(a). Compression index of remolded clay as a function of the liquid limit (Fujikawa & Takayama 1980b).

Figure 56(b). Compression index of undisturbed clay as a function of the liquid limit (Fujikawa & Takayama 1980b).

which is shown in Figure 56(a). This relationship is almost identical with the Eq.(4) by Skempton. The

compression index for undisturbed Ariake clay, with a sensitivity of 5 to 50, is plotted against the liquid

limit in Figure 56(b), where the compression index scattered in a wide range at a given liquid limit

above the correlation proposed by Skempton (1944) for undisturbed clays. This result indicates that the

compression index for undisturbed clay is not exclusively represented as a function of the liquid limit. Taking into account that soil structure affects the compressibility of undisturbed clay, the ratio of the

compressibility for undisturbed clays, C_c , to that for remolded clays, C'_c , was found to be represented as

a function of sensitivity (Fujikawa & Takayama 1980b)

which is shown in Figure 57. Combining Eq.(6) with Eq.(7) gives

Figure 57. The ratio of compression index for undisturbed clay, C_c , to that for remolded clay, C'_c , as a function of

sensitivity (Fujikawa & Takayama 1980b).

Park and Koumoto (1998) proposed the following equation for the compression index of sensitive

Ariake clay.

Thus, the compression index for undisturbed Ariake clay with a sensitivity of 5 to 50 is represented as

the function of the liquid limit and sensitivity, indicating that the effect of soil structure has to be taken

into account when dealing with the compressibility of highly sensitive clay.

4.5.2 Consolidation coefficient

The coefficient of permeability, k , and coefficient of volume compressibility, m_v , would be controlled

by the chemical and physical properties of soil materials such as clay mineral composition, particle size,

pore-water chemistry, exchangeable cations, and organic matter etc. Hence consolidation coefficient, C_v ,

represented by $k/m_v \gamma$ would be also controlled by these factors. Taking into account that these factors are

reflected in the liquid and plastic limit of soil materials, the coefficient of consolidation versus plasticity

index relationship for Ariake clay from Isahaya (sample T, Fig. 1) and Yokoshima (sample Y0) is shown

in Figure 58. The coefficient of consolidation for samples under a normally consolidated state tends to

decrease with increasing the plasticity index while that

for the samples under an overconsolidated state

was almost constant for a change in the plasticity index.

4.5.3 Secondary consolidation

Secondary consolidation generally gives an approximately linear relationship between void ratio (e) and

$\log$ time at least over the first one or two $\log$ cycles of time which is usually the period of practical interest.

The rate is therefore commonly expressed by the coefficient of secondary compression defined as:

Mesri & Godlewski (1977) made a detailed study of the relationship between $C_{\alpha e}$ and compression index,

C_c , for natural soils which show $C_{\alpha e}/C_c$ values generally equal to 0.05 ± 0.02 (Ladd et al. 1977). Figure 59

shows the relationship between $C_{\alpha e}$ and C_c determined for undisturbed Ariake clay. The $C_{\alpha e}/C_c$ values

are mostly in a range of 0.03 to 0.05, which falls in a range proposed by Ladd et al. (1977). Figure 60

shows that there is a linear correlation between the $C_{\alpha e}$ value and the e_s value, the void ratio at which

secondary compression initiates (Kanayama et al. 2001).

Mesri & Godlewski (1977) concluded that the mechanisms responsible for volume changes during

secondary compression are basically the same as those during primary consolidation. Thus all mecha

nisms of volume change (deformation, slippage and reorientation of particles; changes in double layer θ 20 40 60 80 100 10 1 10 2 10 3 10 4 C o n s o l i d a t i o n c o e f f i c i e n t c_v (cm²/d) Plasticity index I_p T Y0 Overconsolidated Normally consolidated

Figure 58. Relationships between coefficient of consolidation and plasticity index for clays. θ 1 2 0 0.05 0.10 $C_{\alpha e}$ C_c $C_{\alpha e}$ θ . 0 5 C_c $C_{\alpha e}$ θ . 0 7 C_c $C_{\alpha e}$ θ . 0 3 C_c Single drainage T-2 T-3 T-4 T-5 T-6 T-7 T-8 T-9 Double drainage T-1 T-2 T-3 T-4 T-5 T-6 T-7 T-8 T-9 T-10

Figure 59. Correlations between the coefficient of secondary compression, C_{α} , and compression index, C_c , for

undisturbed clays (Kanayama, et al. 2001).

thickness; distortion of 'adsorbed' water films; etc.) that can occur during changes in effective stress

(i.e. consolidation) can also operate during changes in time at constant effective stress. Equation (11) was proposed to estimate the magnitude of settlement due to secondary consolidation,

assuming that the shape of a time versus settlement curve accords with that of Terzhagi's theoretical

curve up to around 80% consolidation degree and beyond that the magnitude of settlement increases

linearly with time in a logarithmic scale (Ministry of Land Infrastructure and Transport, 1989). 1 2 3 4 0 0.02 0.04 0.06 0.08 0.10 C_{α} e e s T-1 T-2 T-3 T-4 T-5 T-6 T-7 T-8 T-9 T-10 T-11 C_{α} 0.0265 e s 0.00618 r 0.961

Figure 60. A correlation between the coefficient of secondary compression, C_{α} , and the void ratio, e_s , at which

secondary compression initiates (Kanayama et al. 2001). .

Figure 61. Sliding failure of slope at a site of a canal (Takayama 1995).

where S_s = the magnitude of settlement due to secondary consolidation, C_{α} = coefficient of secondary

consolidation, t = time, t_0 = time when secondary consolidation starts, and h = clay layer thickness. The

C_{α} value is estimated based on equation (12) proposed by Mesri & Godlewski (1977).

5 REDUCTION IN STRENGTH IN SLOPE FAILURE AND REGAINING OF STRENGTH

Irrigation canals for agriculture with a depth of 2 to 3 m have been constructed in open fields around

Ariake Bay. The stability of slopes was assessed at a site of the canal where fill was placed on the bank

of the canal (Fig. 61). The geotechnical properties of the site are shown in Figure 62. The natural water

content was much higher than the liquid limit throughout the depth, and the shear strength and sensitivity

exhibited the range of 9 to 18 kPa and 20 to 40 (not shown), respectively. The soil material derived from

sand stone was used for filling, which possesses a natural water content of 32%, and the liquid and

plastic limit of 44 to 57% and 30 to 34%. Concrete piles were installed at the toe of the slope in the canal

before filling.

The slope of the canal failed during rainfall two months after the filling work was completed. During

rainfall the geotechnical parameters of the filling material changed from 85 to 96% in saturation degree

and 17.6 to 18.1 kN/m³ in total unit weight. This led to the reduction in the cohesion of the material from

9 to 7 kPa and the friction angle from 6.5 ° to below 4.5 °. The strength parameters of filling material

before and after failure were assessed using unconsolidated undrained shear tests.

Figure 62. Geotechnical properties of a clay profile at the site where sliding failure occurred (Takayama 1995).

Figure 63. Changes in the shear strength acquired with time after sliding failure occurred (Takayama, 1995). Figure 63 shows the profile of the undrained shear strength determined by unconfined compression

tests, strain at failure, and modulus of deformation with depth before and after the

failure occurred. The unconfined shear strength parameters were determined for borehole samples

immediately after the failure of the slide occurred. Lowering of the shear strength was observed in

the zone of slip plane at a depth of 4.5 to 5 m. Increases in the strain at failure and decreases in the

modulus of deformation were also noticed in the zone of the slip plane, which indicate that the soil

structure in this zone was disturbed by shearing due to failure of the slope.

The undrained shear strength of the borehole samples was determined periodically after failure of the

slope. Regaining of the shear strength with time was observed, and the strength was fully recovered after

13 months as a result of thixotropy.

6 SUMMARY AND CONCLUSIONS

The present paper provided the integrated information on the geological setting, the mineralogy, the

depositional and post-depositional geochemistry, and the overall geotechnical properties for Ariake Bay

sediment. The conclusions drawn from this study is summarized in the following.

(1) The recent history of the formation of Ariake clay sediments can be constructed based on the depth and width of Ariake Bay's outlet and sea-level changes over the past 125,000 years.

(2) The main clay minerals in the Ariake clay deposits are smectite, illite, kaolinite, vermiculite, chlorite and halloysite. The percentage of smectite decreased with depth while the percentage of other clay minerals was almost the same throughout the depth. The smectite in the sediment was found to be of low-swelling type from the sediment volume tests.

(3) For the non-salt-leached clay sediments at all the reported sites except Yamaashi, pore water salinity (Cl⁻ concentration) varied corresponding to the sea level change, from brackish at the base to marine, and then brackish toward the surface of the deposit. For the salt-leached sediment at Yamaashi, pore water salinity (Cl⁻

- concentration) was less than one tenth of that for the non-salt-leached sediments. The higher iron oxide contents in the zones near the surface and below 14 m for non-salt-leached sediments indicated that these two zones had been subjected to weathering brought about by an exposure of the sediment surface to the atmosphere under lower sea levels.

(4) The distribution pattern of clay fraction was similar in the northern and eastern coasts but was different in the western coast. These can be explained in terms of the sea water movement in Ariake Bay. The distribution pattern of the liquid limit of soil materials was clearly different between the upper and deeper zones.

(5) The main controlling factors of the liquid limit of soil materials was smectite content in clay fractions and pore water salinity, both of which contribute to enhance the liquid limit.

(6) The shear strength exhibited linear increase with depth for both the upper and deeper zones, with a greater rate of increase for the deeper zone. This was demonstrated by the magnitude of the S_u / p values, giving greater S_u / p values for the deeper zones in the non-salt-leached clay profiles.

(7) No correlation was observed between the S_u / p ratio and plasticity index for Ariake clay and the S_u / p ratio was much greater than the Skempton-Bjerrum correlation line in a broad range of plasticity index.

(8) In strength anisotropy tests, active shearing gave higher shear strength than passive shearing, and the active shearing for the sample with $\alpha = 45^\circ$ gave the highest shear strength. A clear difference in the extent of strength anisotropy was observed between the samples of 1 to 12 m and those of 13 to 17 m depth; the samples in the deeper zone indicated greater extent of anisotropy compared with those in the upper zone.

(9) Development of sensitivity less than 50 in clay sediment can be explained in terms of slow load increase and minimal consolidation, and thixotropic processes of clays, while the sensitivity greater than 50 was found to develop by the leaching of pore water salinity.

(10) The compression index was correlated with liquid limit for remolded Ariake clay, which was almost identical with that proposed by Skempton. The compression index for

undisturbed Ariake clay, however, was far above the Skempton's correlation line, and was represented as a function of the liquid limit and sensitivity.

(11) The coefficient of consolidation for undisturbed clays under a normally consolidated state tends to decrease with increasing plasticity index while that under an overconsolidated state was almost constant for a changing in the plasticity index.

(12) The ratio of the coefficient of secondary compression, $C_{\alpha e}$, to the compression index, C_c , for undisturbed Ariake clay was mostly in a range of 0.03 to 0.05, which falls in a range proposed by Ladd et al (1977). A linear correlation exists between $C_{\alpha e}$ and e_s , the void ratio at which secondary compression initiates.

(13) Although increases in strain of failure and decreases in the modulus of deformation were observed for soil materials after failure of slope at a site of the canal, the shear strength of the soil materials was fully recovered after 13 months of the slope failure.

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Characterisation and Engineering Properties of Natural Soils - Tan, Phoon, Hight & Leroueil (eds) © 2007 Taylor & Francis Group, London, ISBN 978-0-415-42691-6

Characterisation and engineering properties of Troll Clay

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ABSTRACT: The Troll site is underlain by a thick deposit of uniform marine clays and glacial tills.

Though most of the work reported here was related to the development of a large CONDEEP concrete

gravity structure, which at the time of its installation was the largest in the world, the ground conditions

encountered apply across a wide area of the Norwegian Trench. Five soil units have been identified

and the soil properties and engineering characteristics are consistent with their geological origin. Most

emphasis is given to the upper two units, i.e. to about 74 m depth. A large range of tests were undertaken

both in the laboratory, on good quality thin walled piston or pushed samples, and in situ including CPT.

The soil properties show good agreement with well - known correlations.

1 INTRODUCTION

Troll is the largest gas field offshore Europe with proven reserves of more than 1300 billion standard

m³ of gas. The field was discovered by Shell in 1979; production with export to the European continent

started in 1996 and is expected to continue for several decades. The Troll field lies in the so-called

Norwegian Trench in water depth in range 300-330 m, the distance to the Norwegian coast is about

65 km. Since the first soil investigation was carried out

in 1983 a large number of soil investigations

have been carried out especially for the large CONDEEP gravity base structure (GBS) that was installed

in 1995, in the so-called East Troll field. Fig. 1.1 shows the location of the Troll East field. The soils

in the upper about 75 m are lightly to moderately overconsolidated due to aging, and are very uniform

across the Troll area. Below about 75 m the soils are overconsolidated due to ice loads with high OCR.

Most of the soil investigations were carried out from the soil drilling vessel 'Bucentaur' using techniques

that are described by Amundsen et al. (1985). By & Skomedal (1992) presented a comprehensive review

of the soil parameters that were used for the foundation design of the CONDEEP gravity base platform.

Table 1.1 summarizes the main soil investigations carried out for the GBS.

Including all regional investigations and studies performed at Troll West, the numbers in Table 1.1

above would be almost doubled. In addition, other in situ tests have been utilized: vane shear, push-in

pressuremeter, hydraulic fracture, dilatometer and BAT probe tests. Most of the equipment and test

procedures have been described by Amundsen et al. (1985).

The work of By & Skomedal (1992) is a key reference for this paper. Fig. 1.2 gives a summary soil

profile for Troll, and Table 1.2 summarises the soil stratigraphy.

The reader is also referred to the parallel paper presented to this conference by Uzielli et al. (2006)

who perform a statistical characterisation of data from the Troll site.

Figure 1.1. Location map, Troll East.

Table 1.1. Summary of main soil investigations performed close to final platform location. Triaxial tests DSS tests Borings Seabed Oedometer Resonant

Year (m) CPT, m Static Cyclic Static Cyclic tests column/G max

1984, area 1 315 125 38 8 36 18 41 -

1987 400 200 28 16 13 13 - 6

1988 330 455 14 12 7 8 4 13

1989 360 415 22 18 18 18 21 31

Total 1405 1195 102 54 68 57 66 50

2 ENGINEERING GEOLOGY

2.1 Depositional environment

The discussion here follows on the descriptions of the various soil units as detailed on Table 1.2 above.

and NGI (1989).

General background

A detailed geological background of this area of the North Sea is presented by Green et al. (1985). These

authors outline four possible geological models for the area and present a new alternative which is very

similar to what they term the "NGI / Geoteam" model. The designation of the various units in the Green

et al. (1985) paper are, for all practical purposes, the same as presented here. Andersen (1989) based his

work on a 37.6 m core obtained in the centre of the Norwegian Trench at the Troll field and thus focuses

his work on the upper two units.

Unit I (0-16.5 m)

Unit I is a glaciomarine sediment which was deposited immediately after the deglaciation of the area

beginning some 13,500 years B.P. Its deposition is thought to have occurred in a low energy marine

environment while the sea level increased and water became deeper. Approximately 8000 years B.P.,

Figure 1.2. Soil profile (from By & Skomedal 1992).

Note: s_u = undrained shear strength, q_{net} = net CPT cone resistance, w = water content, I_p = plasticity index,

S_t = sensitivity, OCR = overconsolidation ratio, $\phi' =$ effective friction angle and $a =$ attraction.

corresponding to about 1.5 m depth, the English Channel opened and this led to the onset of significant

Atlantic influences. Sedimentation rate was then severely reduced and this may have produced higher

compaction of the sediments and thus the higher measured CPT values. The material in Unit I is generally

well laminated probably, due to cyclic depositions following variable meltwater discharge.

Table 1.2. Stratigraphic boundaries (NGI 1989).

Depth interval (m)	Unit	Soil description
--------------------	------	------------------

0.00-16.50	I	Clay, very soft to firm, high plasticity, slightly overconsolidated
------------	---	---

16.50-44.00	IIA	Clay, sandy to very stiff, medium plasticity, slightly overconsolidated, many gravels and occasional stones
-------------	-----	---

44.00-64.00	IIB	Clay, as unit IIA, but very stiff to hard, and slightly more overconsolidated
-------------	-----	---

64.00-74.00	IIC	Clay, as units IIA and IIB but less overconsolidated
-------------	-----	--

74.00-101.00	IIIA	Clay, sandy, very hard, highly overconsolidated, layers of fine sand and silt, many gravels and stones, low plasticity
--------------	------	--

101.00-135.00	IIIB	Clay, very hard, overconsolidated, many gravels and stones, high plasticity
---------------	------	---

135.00-201.00 IV Clay, very hard, overconsolidated, a few gravels, no stones or boulders, high plasticity

201.00-219.00 V Clay, tertiary, very hard, overconsolidated, homogeneous, very high plasticity

Unit II (16.5 m-74 m)

Unit II has been sub-divided into three zones (IIA, IIB and IIC) based mainly on water content, undrained

shear strength and degree of overconsolidation. Unit II consists of a series of glacial tills deposited while

the ice sheet advanced and then subsequently when it melted. This deposition occurred between 13,000

years and 20,000 years B.P. As the ice-sheet moved across the area, it subjected the soils to "dry-based"

erosion followed by "wet-based" till deposition. Subsequently, deposition occurred from suspension and from floating ice in a quiet and stable basin

without bottom current activity. The degree of overconsolidation of Unit II is relatively low, therefore

excluding any major loading by a glacier, and it is thought that the glacier was afloat during deposition

of the unit. It is likely that the glacier periodically "touched down" causing the higher consolidation of

Unit IIB compared to Unit IIA. It is likely that Unit IIC is at least partly a lodgement till, though its

degree of overconsolidation and strength are lower than expected. There appears to be an undulating interface between Unit I and Unit II. This surface has been interpreted

to have been generated by iceberg ploughing. This ploughing has contributed to the homogenisation of

the sediments. In geological terms, Unit II is sometimes termed a diamicton, i.e. it has a relatively liner

grading curve with particles of size ranging from clay to boulders.

Unit III (74 m-135 m)

Unit III is similarly sub-divided into two zones. It is much more overconsolidated than Units I or II. Again

the unit is thought to be a series of glacial tills which may have been deposited by the same ice-sheet

as that responsible for Unit II or by one from a previous glaciation. The top of Unit III is an erosional

surface which is easily identified on seismic sections. This surface was caused by glacial advance which

removed an unknown quantity of the underlying strata. The increase in consolidation from Unit II to

Unit III could be due to a previous sediment load or glaciations. Like Unit II, it is possible that the

lowest sub-unit comprises a lodgement till, and that the upper zone was formed as a melt out till. The

soils of Units III and IV are thought to be between 750,000 and 1 million years old (i.e. from the early

Quaternary).

Unit IV (135 m-201 m)

The top of Unit IV is an irregular erosional surface, with evidence of some channels. Unit IV is thought

to represent glaciomarine sediments deposited in progressively deepening water. These sediments are

probably very similar to Unit I soils. They may represent offshore deposits resulting from an earlier

glaciation and are probably Tertiary sediments. The soils in Unit IV contain a high percentage of clay,

indicative of a quiet marine environment. It seems the maximum depth below sea-level during deposition

was of the order of 50 m.

Unit V (201 m-220 m)

On seismic profiles the base of Unit V is marked by a very prominent angular unconformity. Above the

unconformity the sediments are a sandy gravelly clay suggesting they have been deposited in either a

fluvial or shallow marine environment. Subsequently an increase in sea-level resulted in the deposition

of the remainder of the unit, which becomes a sandy silty clay as the water depth increased. The age of

these sediments is 25 million to 40 million years. Their mineralogy is very different from Unit IV, see Table 2.1. Summary of stress history. Depth (m) Unit Range of OCR Description 0-74 I and II 1.25-1.55 Slightly overconsolidated 74-220 III to V 2.35-7 Highly overconsolidated

Table 3.2, with a relatively high smectite (a swelling mineral) content being recorded. The top of Unit

V/base of Unit IV are marked by an irregular, apparently erosional surface.

Beneath Unit V

The sediments beneath Unit V are of Tertiary age and dip gently to the west at an angle of less than 5° .

2.2 Source of material

Green et al. (1985) suggest that the stratification of the Quaternary sediments indicates two main direc

tions of transport, one across the Norwegian Trench and the other parallel to the Trench. Transport across

the Trench was glacial, while transport parallel to the Trench was more likely fluvial or glaciofluvial.

The clay mineralogy (see Table 3.2) indicates two sources for the sediments: (i) material from chlorite

containing crystalline rocks in Scandinavia and (ii) material reworked from kaolinite rich sediments and

bedrock which are found in the North Sea area.

2.3 Post depositional processes

Each of the Units III to V have been subjected to significant post depositional processes, mostly erosion

and ice loading, as described above. Units I and II (i.e. the upper 74 m) have not been subjected to

significant ice loading, but ageing has caused some apparent preconsolidation effect. This effect has

been noted in many other marine clays, for example at Onsøy as reported by Lunne et al. (2003).

2.4 Stress history

The stress history as evaluated from constant rate of strain (CRSC) oedometer tests, carried out to the

procedures detailed by Sandbækken et al. (1986) on high quality piston samples, can be summarized as

in Table 2.1.

In situ effective overburden stress and preconsolidation stress are presented in Section 6.

3 MATERIAL COMPOSITION

3.1 Grain size distribution

Figure 3.1 gives typical grain size distribution curves for soil units I to IV (i.e. uppermost 200 m). Test

were carried out according the procedures described by Moum (1965).

In Table 3.1, a summary is given of average values of percentages by weight of the clay and granular

fraction in the various soil units. It can be observed that the clay content in these units decreases with

depth.

3.2 Form of clay fraction

X-ray diffraction analyses were performed on both the clay fraction ($<2\text{ }\mu\text{m}$) and the sand fraction

($>60\text{ }\mu\text{m}$) (NGI 1984). Results for the average values from

two sets of analyses are shown in Tables 3.2

and 3.3 respectively.

For the clay fraction analyses there is a clear difference in the results between Units IV and V, as can

be seen from Table 3.2. Unit IV shows values typical for other North Sea sites, whereas Unit V contains

mostly smectite, which is a swelling mineral. No data are available for the sand fraction in Units IV

and V.

Figure 3.1(a). Grain size distribution curves for Units I and II.

4 STATE AND INDEX PARAMETERS

4.1 Water content and Atterberg limits

Figure 4.1 shows water content (w) and plastic (w_P) and liquid (w_L) limits versus depth. In soil Unit I, water content ranges from 45 to 90% and is close to the liquid limit. The plasticity

index (I_p) is on average about 37%. There is a marked drop in water content in Unit II, where it ranges

from 26% to 15% while plasticity index is on average about 20%. For Units IIB, IIC and lower units

w is close to w_P . In Unit III water content ranges from 12% to 19% while I_p ranges from 13% to 28%.

Figure 3.1(b). Grain size distribution curves for Units III and IV.

In Units IV and V water content is more variable and ranges between 15% and 37%. Average plasticity

index for these two units is 32% and 50% respectively.

The trends in the relationship between plasticity and water content can be seen more clearly on the

plot of liquidity indeed (I_L) with depth on Fig. 4.2. I_L is defined as:

Table 3.1. Summary of grain size distribution. Average
Average

Depth (m)	Unit	No. of tests	<2 μm (%)	>60 μm (%)	Remarks
0-16.5	I	3	44	3	
16.5-44	IIA	4	26	34	
44-64	IIB	2	22	38	
64-74	IIC	1	22	41	
74-135	III	7	7-34	30-58	*
135-201	IV	5	37-69	3-14	*
201-220	V	4	36	21	

*The range is given due to the relatively large variation.

Table 3.2. Average values for mineralogy of clay fraction
(<2 μm).

Mineral	Unit I	Unit II	Unit III	Unit IV	Unit V
Illite (%)	54	47	50	56	20
Kaolinite (%)	19	20	22	14	20
Smectite (%)	13	13	10	13	60
Chlorite (%)	6	10	6	12	-
Quartz (%)	2	2	Minor	Minor	-
Feldspar (%)	2	2	Minor	Minor	-

Calcite Minor Minor Minor Minor - Table 3.3. Average values
for mineralogy of sand fraction (>60 μm). Mineral Unit I
Unit II Unit III Quartz (%) 65 65 70 Feldspar (%) 17 20 12
Chlorite (%) 10 5 18 Pyrite (%) 8 5 -

Figure 4.1. Water content, plastic and liquid limits.

Figure 4.2. Liquidity index as a function of depth.

Figure 4.3. Plasticity chart.

It can be seen that I L decreases from about 1.0 in Unit I
to 0.2 at the base of Unit IIC. The low, and

occasionally negative, values in Units III and below are indicative of overconsolidated material with

higher undrained strength than the overlying material.

Plasticity data is also presented on the standard "A" line plasticity chart on Figure 4.3. A general trend

is for the data to fall above and parallel to the A-line indicating "clay" like behaviour, with soils being

classified as either CL or CH (inorganic clays of medium to high plasticity, according to ASTM D2487

or BS5930 "Site Investigations").

4.2 Soil unit weight

Figure 4.4 shows the submerged unit weight versus depth. Both measured and calculated data (based on

w , γ_s , γ_w and assuming full saturation, $S_r = 100\%$) are presented. In Unit I it varies from 6 to 7.6 kN/m³,

while in Unit II it ranges between 9 and 11.8 kN/m³, with a systematic increase with depth. Submerged

unit weight values are highest in Unit III and range mostly between 11 and 13 kN/m³. Values for Units

IV and V are more scattered but are typically about 9 kN/m³.

Figure 4.4. Submerged unit weight versus depth. Table 4.1. Unit weights of solid particles. Depth (m) Unit No. of tests Average γ_s (kN/m³) 0-16.5 I 2 26.8 16.5-44 IIA 3 26.6 44-64 IIB 2 26.5 64-74 IIC 1 26.7 74-135 III 1 26.7 135-210 IV 3 27.0 210-220 V 2 26.1 The vertical effective stress vs depth profile is included in Fig. 6.1.

4.3 Unit weight of solid particles

Average unit weight of solid particles data (γ_s) are summarised in Table 4.1. In general it can be seen

there are only minor differences between the various units.

5 STRUCTURE

The macrofabric of the material in Unit I is generally well laminated probably, due to cyclic deposi

tions following varying meltwater discharge. A discussion of the structure is also incorporated into the

interpretation of the 1D compression data in Section 6.10.

6 ENGINEERING PROPERTIES

6.1 In situ vertical stress and yield stress (or apparent preconsolidation stress)

The variation of in situ effective overburden stress (p'_{θ}) with depth is presented together with values of

preconsolidation stress (p'_c) on Fig. 6.1. This is based on a best estimate of total unit weights as presented

in Section 4.2.

Figure 6.1. Preconsolidation stress (a) measured values and (b) design profile adopted (from By & Skomedal 1992).

Note: different scales used in the two plots.

Values of estimated preconsolidation stress (p'_c) or vertical yield stress from oedometer tests are shown

on Figure 6.1a. These tests were carried out mostly on either wireline thin-walled piston tube samples or

thin-walled pushed samples, which were obtained using recognised offshore techniques (see Amundsen

et al. 1985). For these tests, axial strain (ϵ_a) measured at p'_{θ} were typically between 2.5% and 3.5%

indicating the samples were of "good" to acceptable" quality according to Kleven et al. (1986). These ϵ_a

values are equivalent to $\Delta e/e_{\theta}$ in the range 0.04 to 0.08 (see detailed analysis of sample quality presented

in Section 7).

Both the Janbu (1970) and Casagrande (1936) techniques were used to derive the results. In addition

an empirical correlation, established at NGI, based on s u

from triaxial tests, p'_{θ} and I_p , was used (Andresen et al. 1979).

Several higher values of p'_c were obtained for Unit IIIA as can be seen in the design profile chosen

for p'_c on Figure 6.1b. A line representing p'_{θ} is also shown in Figures 6.1a and 6.1b.

6.2 OCR

Based on the p'_c values discussed above, overconsolidation ratio (OCR) values were calculated. These

are presented together with a recommended design profile, on Figure 6.2. A summary of OCR values is

given in Table 2.1.

6.3 In situ lateral stress

A profile of earth pressure at rest coefficient K_{θ} against depth is presented in Figure 6.3. This has been

derived mostly from an empirical relationship between OCR and I_p from Brooker & Ireland (1965). Some

Marchetti dilatometer tests (DMT) were also carried out, and the results interpreted for K_{θ} according to

Marchetti (1980). K_{θ} values varied between 0.6 and 0.8 and 0.6 and 0.7 for the depth intervals 46 m to

48 m and 65 m to 67 m respectively, see Figure 6.3. These values are consistent with those obtained by

the empirical approach.

Figure 6.2. OCR vs depth.

6.4 Small strain shear modulus, $G_{\max}$ (By & Skomedal 1992)

Small strain shear modulus ($G_{\max}$) was evaluated from a comprehensive database of labora

tory tests comprising, resonant column piezoceramic bender element (Dyvik & Madshus,

1985, Dyvik & Olsen 1989) and small strain measurement test

results. Similarly G_{max} was measured in

situ using the seismic cone. These data are presented on Figure 6.4. When deriving design values, most emphasis was placed on the seismic cone and the piezoceramic

bender element tests as these two types of tests are the only ones which provide a direct measurement of

G_{max} based on the shear wave velocity. As can be seen from Figure 6.4, values from these two tests were

of the same order of magnitude. Resonant column tests yielded significantly lower G_{max} values. Scatter

in the test results increases with depth reflecting the increasing effects of sample disturbance caused by

tube sampling and higher stress relief. Typical average G_{max} values increase from about 25 MPa in Unit

I to 125 MPa in Unit IIA.

Figure 6.3. K_0 versus depth.

6.5 Stress strain behaviour in 1D compression

Typical 1D consolidation stress strain curves are shown in Figure 6.5. These data were obtained using a

thin walled piston tube sample from the 1989 investigation and from the new deep water sampler (see

Section 7) in 2005. Both samples are from Unit I at about 10 m. Data are presented in the conventional

$\log \sigma'_a$ versus axial strain (ϵ_a) format (Fig. 6.5a) as well as in linear scale (Fig. 6.5b). In Fig. 6.5b, values

of the constrained modulus M ($=\Delta\sigma'_a/\Delta\epsilon_a$, Janbu, 1963, 1970) versus ϵ_a are also shown.

The curves show the classic response of slightly structured, lightly overconsolidated clay. On initial

loading the response is relatively stiff. Constrained modulus values increase rapidly, reaching a maximum

near p'_0 , and then decrease to a minimum value close to p'_c . The p'_c value can be seen clearly defined

in all three plots, confirming the good quality of the samples. Post yield the constrained modulus

values increase more or less linearly with stress (the slope of this part of the plot being the modulus

number m).

Figure 6.4. G_{max} values from lab testing and in situ testing using seismic cone (By & Skomedal 1992).

6.6 Constrained modulus, M

Values of constrained modulus, M , corresponding to a stress in the range $p' = 0 + \Delta p < p' = p'_c$, are shown on

Figure 6.6. Here Δp corresponds to the increase in stress above the in situ overburden stress with the

entire stress increment remaining within the overconsolidated zone. Figure 6.6a focuses on the results

for Units I and IIA, whereas Figure 6.6b reports on data for Units I to III. In general the values show an

approximate linear increase with depth following a similar pattern to G_{max} . However the values of M are

significantly lower, being on average typically 2 MPa for Unit I and 10 MPa for Unit IIA. The increasing

scatter with depth reflects the influence of sample disturbance and stress relief.

6.7 Compressibility in normally consolidated range

Compressibility in the normally consolidated zone is defined by the slope of the M versus σ'_a a line post

yield (i.e. modulus number, m) and by the intercept of this line on the x-axis (reference stress, p'_r) (Janbu

1963, 1970). Some examples of these values for Units I and IIB are shown in Figures 6.7a and 6.7b

10 100 1000 10000 Effective Axial Stress, σ'_a (kPa)

36

30

24

18

12

6

0

A x

i a

l s

t r a

i n ,

e a

(%

)

1E-011 1E-010 1E-009 1E-008 Coefficient of Permeability, k
(m/s) DWS 101 7G1 (10.30m) 8901 6B1 (10.12m) 1st loading
reloading 0 200 400 600 800 1000 1200 1400 Effective Axial
Stress, σ_a (kPa) 42 36 30 24 18 12 6 0 A x i a l S t r a
i n , e a (%) 0 4 8 12 16 20 24 28 M o d u l u s M (M P
a) DWS101 7G1 (10.30m) 8901 6B1 (10.12m)

Figure 6.5. Typical 1D compression curves, (a) log scale
and (b) linear scale (NGI 2006).

Figure 6.6. Constrained modulus M for stress range $p' \geq p_c$
+ $p_c < p' \leq p_c$ (i.e. within overconsolidated zone) (a) Units I
and IIA only and (b) Units I to III (By & Skomedal 1992).

Figure 6.7a. Modulus number and reference stress, Unit I
(from NGI 1989).

respectively. These m values of 13 and 20 respectively are
consistent with published correlations between

m and water content. For example Janbu (1970) suggests that if average water content is about 55%

(as for Unit I), then m will be about 14, and if w is close to 20% (Unit IIB), then m will be of the

order of 30. Note that m can be related to the compression coefficient C_c by the expression:

6.8 Permeability and coefficient of consolidation

Results of measurements of co-efficient of permeability both in the horizontal (k_h) and vertical direction

(k_v) are presented on Fig 6.8. A variety of techniques were used to determine the values, e.g. laboratory

permeability and CRSC tests, dissipation tests during piezocone testing and in situ permeometer testing. There is no clear anisotropy of permeability and it can be assumed that $k_h \approx k_v$. Moreover there are no

significant differences between the results from the different techniques or the different analysis routines

used to deduce k from the dissipation tests. Typically, k drops from about 0.1 m/yr. on average for Unit

I to 0.003 m/yr. for Unit IIA. Values for the lower units are more scattered, but range in the same order

of magnitude as for Unit IIA. The coefficient of consolidation (c_v) varies with applied stress in a similar manner to constrained

modulus as described in Section 6.5. Data for the same oedometer tests as in Figure 6.5 are shown on

Figure 6.9. It can be seen that c_v values are high when the stress is less than p'_c , and reach a minimum

close to this value.

Figure 6.7b. Modulus number and reference stress, Unit IIB (from NGI 1989).

A summary of all values of c_v are shown in Figure 6.10. As in the case of permeability a variety

of techniques were used. Again there seems to be no clear difference in the results from the different

techniques, with c_v being typically $4 \text{ m}^2/\text{yr.}$ to $8 \text{ m}^2/\text{yr.}$ for $p' < p'_c$ and $2 \text{ m}^2/\text{yr.}$ to $4 \text{ m}^2/\text{yr.}$ for $p' > p'_c$.

Values of horizontal coefficient of consolidation (c_h) were obtained from special oedometer tests,

by appropriate trimming of specimens, and dissipation tests (Svanø 1982). From the results it can be

assumed that $c_h \approx c_v$.

6.9 Creep behaviour

On the settlement versus log-time curve secondary, compression (or creep) generally appears as a straight

line for the Troll clay. The rate of secondary compression is defined as (e.g. by Lambe & Whitman 1976):

where: $\dot{H}$ = secondary compression and H = drainage path length.

The rate of secondary compression is much higher in the normally consolidated part of the consoli

dation curve ($p'_{\theta} + \Delta p > p'_c$) than in the overconsolidated range ($p'_{\theta} + \Delta p < p'_c$). It should be noted that

p is the change in stress above the in situ overburden stress. In the normally consolidated part of the

consolidation curve the rate of secondary compression is almost independent of effective stress level

(Janbu 1970).

Figure 6.8. Coefficient of permeability (from NGI 1989).

Figure 6.9. Variation in coefficient of consolidation with stress (NGI 2006).

Figure 6.10. Coefficient of consolidation (By & Skomedal 1992). Table 6.1. Summary of C_α values. C_α for stress range C_α for stress range $p'_{\theta} + \Delta p < p'_c$ $p'_{\theta} + \Delta p > p'_c$ Soil unit Depth (m) OC range NC range I 0-16.5 0.015

0.03 II 16.5-74 0.004 0.006 III to V 74-220 0 0

Figure 6.11. Troll field - sedimentation compression curves for Unit I (open circles) and Unit IIA (open triangles)

(from Burland 1990). The time-settlement curves from the oedometer tests indicate C_{α} values between 0.01 and 0.03 in the

range $p' < p'_c$ (i.e. in the normally consolidated zone). Janbu (1970) uses the term "resistance number" (r_s) to describe secondary compression. The

relationship between C_{α} and r_s is given by:

For normally consolidated clay, Janbu (1970) suggested r_s is in the range 100-500. Using the typical

range of e_0 values for Troll clay (0.7 to 1.5) gives C_{α} between 0.01 and 0.06, which is consistent with

the lab measurements. For design, the values summarised on Table 6.1 were adopted. The practical implications of these values are: (a) for Unit I large creep may occur when loading up

to p'_c . (b) Moderate to little creep may occur for Unit II, but for the lower units creep will be negligible.

6.10 Assessment of 1D compression behaviour following framework of Burland (1990)

It is also useful to examine the 1D compression behaviour of the material within the framework proposed

by Burland (1990). The sedimentation compression curves for Troll Unit I and Unit IIA are shown

in Figure 6.11 (taken from Burland 1990). Unit I data (open circles) lies above the sedimentation

compression line (SCL) while Unit IIA (open triangles) lies around the intrinsic compression line (ICL).

This suggests that Unit I was deposited in still water by relatively slow deposition leading to an open

Figure 6.12. Troll - typical oedometer tests for Units I (open circles) and Unit IIA (open triangles) together with

Burland's SCL and ICL (from Burland 1990).

random fabric with high values of void index (I_v). In contrast these data suggest Unit IIA has an orientated

and more reworked fabric with lower values of I_v . Both of these findings are consistent with the geological

background of the materials as discussed in Section 2.1 above.

The results of oedometer tests on samples from the two layers, as shown in Figure 6.12 (again taken

from Burland, 1990) confirms the differences in the depositional environment. In Figure 6.12 the open

circles are from oedometer tests on undisturbed samples of Unit I. The compression curves follow the

well established pattern of falling steeply through the SCL and then flattening off and converging slowly

with the ICL. It can be seen the very significant destructuration is necessary to bring the material to its

intrinsic state. This is likely to be due, in part at least, to its laminated nature. A similar finding was

reported by Long (2003) for Athlone laminated clay from Ireland. In contrast the curves for Unit IIA

remain close to the ICL, confirming the fabric is already well oriented, and that compression in the

oedometer does not change things significantly.

6.11 Sensitivity from index testing

A sensitivity profile, based on fall cone tests, is shown on Figure 6.13. Representative sensitivity values

are as summarised on Table 6.2.

Although no test results are available below 74 m, the data indicate a S_t of about 2. For this hard soil,

a sensitivity value of 2 is reasonable.

6.12 Stress strain behaviour from CAUC tests

Prior to considering the undrained shear strength values from triaxial and other laboratory tests, it is

important to understand the stress-strain response of the various materials. As an example some stress-

strain and stress path plots for triaxial tests on Unit I are presented in Figures 6.14 and 6.15. In the

stress-strain plot, the stress has been normalised by the in situ vertical effective stress (σ'_{v0}). The stress

path is presented in $s' = (\sigma'_a + \sigma'_r)/2$ and $t' = (\sigma'_a - \sigma'_r)/2$ format.

Tests were carried out on thin walled piston samples from the 1989 investigation and on specimens

from the new deep water sampler (as described in Section 7). Samples were first reconsolidated to the

best estimate of the in situ stress, and subsequently sheared undrained (CAUC test). All tests were carried

out according to the procedures describe by Berre (1982).

All of the tests confirmed the good quality of the specimens (ϵ_e/ϵ_0 is in the range 0.04-0.06, see

Section 7). Peak stress occurs at relatively small strain and there is considerable strain softening post

Figure 6.13. Sensitivity from fall cone. Table 6.2. Summary of average sensitivity values from fall cone tests. Soil unit Depth (m) S t I 0-16.5 5.5 II 16.5-74 2 III to V 74-220 2 0 2 4 6 8 10 12 14 16 18 20 22 Axial Strain, ϵ_a (%) 0.1 0.2 0.3 0.4 0.5 Normalised Shear Stress $\tau = (\sigma'_a - \sigma'_r) / 2 \sigma'_{v0}$ DSW101 6A1 (Depth 9.00m) DSW101 8A1 (Depth 11.00m) 8901 4B1 (Depth 6.20m) 8901 7B1 (Depth 12.20m) 8901 9C1 (Depth 15.23m)

Figure 6.14. CAUC triaxial stress-strain response for Unit I (NGI 2006). Shear Stress $\tau = (\sigma'_a - \sigma'_r) / 2$ (kPa) 70 60 50 40 30 20 10 0 0 10 20 30 40 50 60 70 80 90 100 Effective Mean Stress, $p' = (\sigma'_a + \sigma'_r) / 2$ (kPa) DWS101 6A1 (Depth 9.00m) DWS101 8A1 (Depth 11.00m) 8901 4B1 (Depth 6.20m) 8901 7B1 (Depth 12.20m) 8901 9C1 (Depth 15.23m) Line representing $\phi' = 29$ deg. and $a' = 4.5$ kPa ϵ_a (%) Pos. Neg. 0.0 0.2 0.5 1.0 2.0 3.0 4.0 6.0 8.0 10.0

Figure 6.15. CAUC triaxial stress paths for Unit I (NGI 2006).

Figure 6.16. CAUC triaxial stress paths for Unit IIB (from Burland 1990).

peak. The stress path plot is initially near vertical and then the soil shows contractive behaviour and

subsequently follows along the Mohr-Coulomb failure line.

The behaviour of Units IIA and IIC during CAUC testing is similar to that of Unit I, i.e. exhibits

contractive behaviour. However Unit IIB and Units III to IV show dilative behaviour on shearing. This

is a characteristic of stiff overconsolidated clays. An example for Unit IIB is shown on Figure 6.16.

Figure 6.17. Undrained shear strength from (a) triaxial compression 0-40 m, (b) triaxial compression 40 m-80 m,

(c) triaxial extension and (d) direct simple shear (from By & Skomedal 1992).

6.13 Undrained shear strength from laboratory tests

Values of undrained shear strength (s_u) in triaxial compression (CAUC), triaxial extension (CAUE) and

direct simple shear (DSS) are shown in Figure 6.17. See Berre (1982) and Bjerrum & Landva (1966) for

details of these test techniques. All specimens were consolidated to estimated in situ stress. Undrained

shear strength was taken at 10% axial strain or lower if a definite peak occurred. Similarly, in DSS tests

the shear strength was taken at 15% shear strain or lower. In practice, the actual failure strains were lower

than 7.5% axial strain and 10% shear strain, except for specimens from Units IIB and III. Selected design lines for each test type are also shown in Figure 6.17. In general each profile shows

a clear increase in strength with depth. Typical s_u (CAUC)

values for Units I and IIA are 25 kPa and

85 kPa respectively. There is more scatter with the deeper tests probably, reflecting increasing sample

disturbance.

Figure 6.18. Normalised undrained shear strength (s_u / p'_0) from CAUC tests (from NGI 1989).

A typical s_u / p'_0 plot, i.e. for Units I and IIA from the CAUC tests is shown on Figure 6.18. On average

s_u / p'_0 decrease from about 0.45 at the top of Unit I to about 0.36 at its base, and remains reasonably

constant throughout Unit IIA.

Undrained shear strength anisotropy is best modelled in NGI's opinion from triaxial compression tests

in the "active" zone of a potential failure surface, triaxial extension strength in the "passive" zone and Table 6.3. Summary of strength anisotropy ratios. Soil unit Depth (m) s_{DSS} / s_{CU} s_{EU} / s_{CI} 0-16.5 0.85 0.63 IIA 16.5-44 0.85 0.71 IIB 44-64 0.75 -

direct simple shear in the horizontal shear zone of a potential failure surface. Typical strength anisotropy

ratios for the Troll clay are summarised in Table 6.3.

6.14 Influence of time after sampling on laboratory results

Lunne & Kleven (1986) studied the influence of time after sampling on laboratory test results by

comparing tests done offshore to those on parallel onshore samples. The objectives of the work were to:

- check the quality of samples offshore,
- facilitate the presentation of design parameters as quickly as possible,
- check the effect of transportation and storage,
- rapidly define stress history and strength characteristic required for specification of onshore tests. Based on a comparison between results onshore and offshore it was

concluded that there were no

significant differences between the two sets of results with regard to:

- w
- $p' - c$
- M
- $s_{DSS u}$
- s_{Cu} Some small differences were noted with the soil unit weights and the shear strain to failure in the DSS

tests. However the overall conclusion was that there were no significant differences between the onshore

and offshore test results.

6.15 In situ undrained shear strength - from CPTU

Typical piezocone penetration data for Units I and IIA are presented in Figure 6.19a in the form of net

cone resistance (q_{net} = corrected cone resistance q_t - overburden pressure, p). These data reflect the high

degree of uniformity of the site area. Typically q_{net} increases from about 0.2 MPa in Unit I to an average

of 1 MPa in Unit IIA. Undrained shear strength values (related to triaxial compression) were derived from the CPT data on

Figure 6.19b, using the formula (Lunne et al. 1997b):

Values of cone factor $N_{kt} = 9$ were chosen for the soil above 16 m and 10.5 for that below 16 m. These

data confirm the usefulness of the CPT for use in predicting s_u .

6.16 Comparison of SHANSEP and recompression tests

Burland (1990) compared the results of SHANSEP tests on Troll clay to standard tests where the samples

were reconsolidated to the in situ stress. His conclusions

were:

- As expected, the SHANSEP test on Unit I was very much less brittle than the CAUC test. The stress path does not rise all the way to the ultimate failure line but bends sharply to the left before reaching it - as a reconstituted soil would do. Thus, according to Burland (1990), by altering the structure of the clay the SHANSEP procedure underestimates both the peak strength and brittleness of the material.

Figure 6.19. (a) Net cone resistance and (b) undrained shear strength from CPT (By & Skomedal 1992).

- For Unit IIA it can be seen that the stress-strain and stress path behaviour from the CAUC test is

reasonably well modelled by the SHANSEP test. It seems probable that the ultimate state of both

samples closely approach the intrinsic critical state line. For a soil whose state lies close to the ICL,

the SHANSEP procedure provides a reasonable normalisation pattern for the natural material since

its structure is not significantly changed during initial consolidation.

6.17 Behaviour of soil under cyclic loading

Cyclic and dynamic soil parameters proved to govern the foundation design of the Troll platform (Hansen

et al. 1992). Consequently, a comprehensive cyclic testing programme was performed.

The cyclic soil data presented, based on cyclic consolidated triaxial and direct simple shear tests, may

be utilized when (Andersen et al. 1988 & 1992):

- Calculating cyclic stability
- Estimating cyclically induced undrained settlements
- Calculating cyclic displacement

Figure 6.20 presents contours of number of cycles to failure as function of normalized cyclic shear

stress, τ_{cy}/s_u , and normalized average shear stress, τ_a/s_u . Failure in these diagrams is defined as either a cyclic shear strain amplitude, γ_{cy} , of 15% or an average shear strain, γ_a , of 15%.

The number of cycles to failure depends upon both the cyclic shear stress amplitude, τ_{cy} , and the

average shear stress, τ_a . When τ_a approaches the undrained static shear strength, changes in τ_a and τ_{cy}

have roughly the same effect on the number of cycles to failure. In the range where τ_a is not close to

failure, the number of cycles to failure is relatively insensitive to the value of τ_a and is mainly governed

by the value of τ_{cy} .

Figure 6.21 presents contours of cyclic and average shear strains after 25 cycles as function of nor

malized cyclic and average shear stresses. We have selected to present values after 25 cycles as 25 cycles

are close to N_{eqv} for a typical storm at Troll East. (N_{eqv} = the number of cycles at the constant cyclic

shear stress that will give the same effect as the actual cyclic load history.)

Figure 6.20. Number of cycles to failure as function of normalized cyclic and average shear stresses (from By &

Skomedal 1992). The average shear strain values presented in these diagrams include the average shear strain due to

the shear stress applied undrained prior to cycling. These diagrams show that in triaxial tests large cyclic shear strains are governing the failure for τ_a/s_u

between -0.05 and 0.20 for Unit I and between 0.0 and 0.25 for Unit II. The corresponding values for

direct simple shear tests are τ_a/s_u less than about 0.2 for large number of cycles and τ_a/s_u less than about

0.35 for small number of cycles for both Unit I and Unit

II. For other combinations average shear strain

is governing the failure. Similarly, contours for normalized average pore water pressure are presented in Figure 6.22 after 25

cycles. The pore pressure values for the triaxial tests have been corrected for the pore pressure increase due

to the change in octahedral normal stresses caused by the undrained shear stress τ_a , applied prior to

cycling such that all the pore pressures are due to change in shear stresses and cyclic loading. The diagrams show that the pore pressure seems to depend upon both the cyclic shear stress amplitude

and the average shear stress.

6.18 Effective stress shear strength parameters

Typical effective strength parameters, interpreted from the Mohr-Coulomb line, for Unit I and Unit

IIB are shown on Figures 6.15 and 6.16 respectively. Note attraction (a) is related to effective cohesion

(c') by:

A summary of the effective strength parameters is given on Table 6.4.

Figure 6.21. Cyclic and average shear strains as function of normalized cyclic and average shear stresses after 25

cycles (from By & Skomedal 1992).

7 ENGINEERING PROBLEMS AND CASE HISTORIES

7.1 Testing out new deep water sampler

With the support from a number of oil companies, NGI established the design basis for a new deep water

sampler with the following project aims:

- sampler to be operational in water depths up to 2000 m,
- samples to be 20-25 m long, pushed in from the sea bottom,

- recovery ratio (length of sample/penetration depth of sampler) to be at least 95%,
- samples to be of quality as good as thin walled piston tube samples taken in the bottom of a borehole.

The background for the design criteria for the new sampler, as well as the definition of the detailed

design criteria, was given by Lunne & Long (2006). The new deep water sampler was manufactured

by the Dutch equipment manufacturer APvandenBerg and subsequently tested out onshore at NGI's test

site in Onsøy in Norway and at Troll. The onshore and offshore testing out was actually performed by

the UK site investigation contractor Lankelma with support from APvandenBerg & NGI (NGI 2006).

Due to the extensive site investigations carried out at Troll (as described in this paper), and the very

uniform soil conditions, it is actually an ideal site for testing out new equipment and techniques offshore.

Figures 6.5, 6.14 and 6.15 above compared the results of CRSC oedometer and CAUC triaxial tests on

samples obtained with the new deep water sampler. It could be observed that the new sampler gave

results that were very comparable with results of similar laboratory tests on samples taken with thin

walled piston tube sampler at the bottom of a drilled borehole.

Figure 7.1 gives results of the change in void ratio due to consolidation of a sample back to the best

estimate of the in situ stress conditions, $\bar{\sigma}/\sigma_o$, where σ_o is the originally in situ void ratio.

Figure 6.22. Normalized average pore water pressure as function of normalized cyclic and average shear stresses

after 25 cycles (from By & Skomedal 1992). Table 6.4. Summary of effective strength parameters. Soil unit Depth

(m) ϕ' (deg.) α (kPa) I 0-16.5 29 4.5 IIA 16.5-44 28.5 8
 IIB 44- 64 29 16 IIC 64-74 28.5 8 IIIA 74-101 28.5 300 IIIB
 101-135 26.5 200 IV 135-201 26.5-30.6 200-300 V 210-220
 30.6 300 Lunne et al. (1997a) suggested that σ_e/σ_v may be
 a good parameter for evaluation sample quality,

and recommended the scale given in Table 7.1 for evaluation
 of sample disturbance. Figure 7.1 indicates that the new
 sampler gives samples that are of at least as good quality
 as thin

walled piston tube samples taken in a carefully drilled
 borehole. In Fig. 7.1 results of laboratory tests on

standard gravity core samples taken at the Fram field (NGI
 1998) are also included. The soil conditions

at the Fram field are very similar to those observed at the
 Troll field. The comparison indicates that the

quality of the new deep water samples is significantly
 higher than the gravity core samples. The results of the
 tests with the new deep water samples obtained at Troll
 gives confidence that the

objectives of the project have been met. It is expected
 that this samples will find considerable use in

future deep water soil investigations.

Figure 7.1. Evaluation quality of various samplers used at
 Troll (from NGI 2006). Table 7.1. Proposed criteria for
 evaluation of sample disturbance. σ_e/σ_v Overcon Very good
 to Good to Very solidation excellent* fair* Poor* poor*
 ratio (1) (2) (3) (4) 1-2 <0.04 0.04-0.07 0.07-0.14 >0.14
 2-4 <0.03 0.03-0.05 0.05-0.10 >0.10 *The description refers
 to use of the samples for measurement of mechanical
 properties.

7.2 Troll gravity base platform

When the Troll CONDEEP gravity base platform was installed
 in 1995 it was the largest of its kind in

the world. The reliable foundation design was a result of
 10-15 years of engineering studies, mainly

carried out in Norway. As detailed by Hansen et al. (1992),
 the key figures of the foundation were:

Geometry, as shown in Figure 7.2:

- Total base area 16596 m²
- Skirt penetration depth 36 m

Loads:

- Submerged weight of platform 2353 MN
- 100 year overturning moment (load coefficient, $\gamma_f = 1.0$) 94,144 MNm
- 100 year horizontal force at sea floor ($\gamma_f = 1.0$) 512 MN

Design life 70 years

Figure 7.2. Troll CONDEEP GBS geometry (from Hansen et al. 1992). Several of the design aspects were unique at the time of developing the Troll field so new calculation

models had to be developed. As outlined by Hansen et al. (1992) these included:

- Penetration of 36 m long concrete skirts, including penetration due to self weight and also due to underbase suction
- Cyclic soil behaviour and long term effects were evaluated using the cyclic soil parameters given in Section 6.16 of above. These parameters gave input to: - Cyclic shear strength for bearing capacity calculations - Excess pore pressure due to cyclic build-up - Cyclic shear modulus to be used for soil spring stiffness analyses due to wave loading
- Settlements, consisting of the following elements: - Immediate settlements - Consolidation settlements - Cyclic settlements - Creep
- Soil reactions needed for the structural design of the platform
- Other design aspects included earth quake loading and platform removal

Input to the above analyses were obtained from the actual soil investigations with comprehensive

laboratory testing, model studies in the laboratory and in

the field, as well as theoretical analyses. Due to the many new geotechnical aspects related to foundation of the Troll CONDEEP platform,

the installation phase was monitored through a comprehensive instrumentation programme. Also the

operational phase has been monitored with measurements of settlements and pore pressures in the

foundation soil. As mentioned in the introduction, the platform was safely installed in 1995, and has

since been a very important part of Norway's infrastructure in connection with export of gas to the

European continent (Fig. 7.3).

8 CONCLUSIONS

This paper has detailed the characteristics and engineering properties of Troll clay from the Norwegian

Trench in the North Sea. Though most of the work reported is related to the development of a large

Figure 7.3. The Troll platform.

CONDEEP concrete gravity structure, the ground conditions encountered apply across a wide area

and the clays are characteristic of those found elsewhere in the North Sea. A detailed geological study

indicated that the soils could be divided in five separate units. It was found that this geological sub

division was consistent with the variation in engineering properties of the soils. The differences in

each unit are most clearly characterised by the water content, unit weight, degree of consolidation and

undrained shear strength. Much effort was directed to understanding the behaviour of the soil under

cyclic loading and these issues are covered in detail in the paper. Some details on the influence of the

soil properties on the design and installation of the

concrete gravity structure are provided. Recently the site has been used for trials of a new deep water soil sampler.

ACKNOWLEDGEMENT

The authors would like to acknowledge a large number of colleagues at NGI who worked on various

aspects of the Troll platform over a period of more than 10 years, from the soil investigations, to

foundation design and monitoring of the installation phase and long term behaviour. The cooperation

with Norske Shell, Statoil and Norwegian Contractors in this period has been very important for NGI.

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Characterisation and engineering properties of

Dublin Boulder Clay

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ABSTRACT: Geotechnical engineering properties and
experience of engineering works in the Dublin

Boulder Clay (DBC), which is the primary superficial
deposit overlying bedrock in Dublin are presented.

This paper attempts to synthesise available information in
parallel with recent work by Skipper et al.

(2005), who provide an updated understanding of the geology
of the DBC. Data from a good number

of sites across the city confirms the uniformity of the
deposit and particularly that of the uppermost

two units, the most important from the point of view of
engineering works. DBC is characterised by a

relatively simple microstructure with low water content,
void ratio and permeability and high density.

Relatively inexpensive MASW surface wave techniques can
give accurate estimates of in situ small

strain stiffness and it was found that DBC is significantly
stiffer than other well-characterised tills. High

undrained triaxial compression strengths were measured and
it appears that simple UU tests on good

quality samples can give much more reliable estimates of s_u
than the currently used SPT tests. s_u in

triaxial extension was found to be surprisingly low. As has
been reported by others the behaviour in shear

of reconstituted DBC compares well with that of the intact
till. Peak and large strain effective friction

angle values are similar.

1 INTRODUCTION

This paper gives geotechnical characteristics of the various formations comprising the Dublin Boulder

Clay (DBC). DBC is the primary superficial deposit overlying bedrock in Dublin, the capital city of

the Republic of Ireland. As a result of the recent boom in the Irish economy, a number of large-scale

civil projects have been completed or substantially advanced. A concentration of such projects and

associated technical data is now available for Dublin. These projects have also provided much more

detailed information on the superficial deposits of Ireland in general.

There are significant economic benefits in understanding the characteristics of the DBC as it underlies

much of the city, see Figure 1. Much of the data presented here is from the Dublin Port Tunnel project

(DPT) northern cut and cover and shaft WA2 sites. This has been augmented by information from other

important developments, as listed on Table 1 and whose locations are shown in Figure 1. These sites

cover an area of some 10 km by 12 km square. As well as adding to the database, information from these

sites will be used to illustrate the consistency of the deposit across the city.

2 BRIEF SUMMARY OF PREVIOUS RESEARCH ON DBC

Much of the early work on the behaviour of Irish glacial till was carried out at Trinity College Dublin

(TCD) under the direction of Prof. Kirwin. Work, such as that of Watson (1962) and Bond (1967), treated

the till as a cohesive material and attempted to explain its behaviour using the concepts which had been 0 1 2 km N Tallaght town centre R i v e r D o d d e r UCD Belfield St

Stephens Green Dail Iveagh Gardens Mater Hospital DPT-WA2
 Ballymun DPT-N cut & cover St James Hospital River Liffey
 Key Site DBC Sand and gravel Bedrock Fill R i v e r T o l k
 a

Figure 1. Location of sites in Dublin and summary of ground conditions.

developed for clay soils. Much of the effort was directed towards understanding the behaviour of the

till as a road sub-grade (e.g. Glynn, 1969, Kirwin and Daniels, 1961). Loughman (1979) studied the

residual shear strength of the till and found that in general the ϕ'_{res} was high ($>30^\circ$) due to the high silt

content of the material. Some later work at TCD (e.g. Hartford, 1985 Hartford and Kirwin, 1985, Hartford, 1987 and Orr

et al., 1981) recognised that the cohesive fraction may in fact largely be rock flour and suggested that

the till actually behaved as a frictional material. Attempts were made to fit the till behaviour into the

critical state soil mechanics framework. It was also recognised in this work that the mode of formation

of the till was unlike sedimentary soils and involved considerable shearing. It was also pointed out that

the traditional concept of a preconsolidation pressure did not apply to this material and that estimates of

preconsolidation pressure based on ice sheet thickness were inappropriate. Some parallel work at University College Dublin (UCD) was summarised in a useful general report

by Hanrahan (1977). Much of the work up to this time was hampered by the lack of high quality samples

Table 1. Summary of sites considered other than Dublin Port Tunnel.

Site	Project	Field work	Special lab data	Reference
Ballymun Raft foundation	for a High quality Geobore-S u r ,	K 0 , stiffness	Authors files	

Northern large tower block, cores from local strain

Gateway gauge transducers s u various tests

Dáil Eireann Retaining structures for High quality
Geobore-S Isotropic Brangan a deep excavation adjacent
cores. Data from compression, (2006) to the Irish
Parliament performance of the oedometer, s u buildings.
retaining walls

Mater Large hospital structure High quality Geobore-S s u
Authors files

Hospital cores MASW geophysics

St. James Platform structures and MASW and cross hole
Donohue

Hospital underground excavation geophysics. et al. (2003)

(LUAS) for light rail system

Iveagh Gardens Portal for Dublin Metro MASW geophysics
Authors files

Tallaght town Commercial development MASW geophysics.

centre with deep excavation Performance of retaining
Farrell et al. walls. Full scale footing (1988 and test
1995b)

UCD Belfield University building Authors files

of the till. Available techniques, such as open drive
(U100) tube sampling, often did not work for the till

because of its high stone content. More recent work at TCD
and UCD has utilised some advances in geotechnical
engineering. These

include the availability of high quality samples of the
till, which were recovered by triple tube rotary

coring techniques, and also more sophisticated laboratory
testing techniques in which transducers were

mounted directly on the specimens. Farrell et al. (1988 and
1995b) report on the results of a study of

the stiffness of the material based on trial footing and pile load test results. They estimated that the

“mobilised” stiffness (E_u) was of the order of 100 MPa. Lehan and Simpson (2000) utilise high quality laboratory data to analyse the behaviour of DBC using

the sophisticated BRICK constitutive model, which accounts for the material’s stress history. Attempts

are also made to fit the behaviour of the DBC into a critical state formulation. This work (see also Lehan

and Faulkner, 1998) also gives some details of the effects of initial stress path on the stiffness of the

material. Similarly Brangan & Long (2001) use the results of high quality laboratory tests to attempt

to understand the behaviour of lightly supported retaining walls in the till. Stiffness values determined

in the laboratory in these two latter studies are consistent with the field observations of Farrell et al.

(1995b) in that $E_u = 100$ MPa coincides with axial strains of the order of 0.01% to 0.1%.

There have also been significant developments in field testing of the till. Small strain stiffness can

now be reliably determined using geophysical techniques such as cross-hole seismic surveys or by using

multichannel analysis of surface waves (MASW), see Donohue et al. (2003). Faulkner (1998) reports

on some CPTU testing in the till.

3 ENGINEERING GEOLOGY

3.1 Background – bedrock

The majority of the Greater Dublin area is contained within the fault-bounded Dublin Basin, underlain

by an argillaceous limestone of Lower Carboniferous age, colloquially known as “Calp”. Nolan (1985)

and Farrell and Wall (1990) indicate that the limestone was

deposited in a shallow sea environment

and that cyclical changes in the water depth and depositional conditions (but still within the shallow

water region) led to marked changes in the rock properties and thickness, variations in the sand and

clay content and inclusions of shale or mudstone layers, occasionally weathered to clay. Marchant and

Sevastopulo (1980) describe it as being a dark grey interbedded and laminated fine-grained limestone

and mudstone/shale in roughly equal proportions, intersected by calcite veins. Nolan (1985) reports the

presence of southwest-northeast orientated faults.

3.2 Quaternary geology of Dublin

Although much work has been carried out on the glaciations that have affected the Irish Sea basin,

relatively few authors have looked at County Dublin, perhaps due to the previously infrequent outcrops

of Quaternary material. The most comprehensive general description of superficial deposits in the Dublin

area was given by Farrell and Wall (1990). The deposits were noted to consist primarily of a lodgement

till containing occasional granular lenses. Farrell et al. (1995a) made the differentiation between the Brown Boulder Clay (known in this paper

as Upper Brown Boulder Clay) and the Black Boulder Clay (known in this paper as the Upper Black

Boulder Clay). Farrell and his co-workers stated that the (Upper) Brown Boulder Clay is a weathering

product of the (Upper) Black Boulder Clay but is broadly similar to it in terms of particle size distribution.

These researchers also briefly note that there appears to be some local variation in colour of the (Upper)

Black Boulder Clay causing it to become brownish. An

important exception to the general quaternary geological history of central Dublin lies in the

existence of a pre-glacial channel just north of the River Liffey, see Figure 1. This was identified by

Farrington (1929). It diverges from the present channel of the River Liffey near Connolly Station and

returns near the mouth of the river. It is not clear whether widely varying sea levels, tectonic movements

or some weakness in the underlying rock gave rise to the channel. However from an engineering point of

view, it has significant importance in that it is generally filled with deposits of glacial and fluvio-glacial

gravels. Depth to bedrock reaches a maximum of about 45 m at the mouth of the river. The mode of formation of the gravel deposits within the channel has not been studied in detail. It

is likely that the deeper deposits were formed by drainage channels either beneath or within the ice.

More recent deposits closer to the surface may be river gravels or river terrace gravels. The engineering

parameters of these deposits are not covered in this paper. Emphasis is placed on the more widespread

and economically important DBC.

3.3 Updated understanding of geology of DBC

Recently Skipper et al. (2005) have provided an updated understanding of the geology of the DBC based

on work for the Dublin Port Tunnel (DPT) project. A simplified schematic of the interpreted stratigraphy

at this site is presented in Figure 2. The following distinct formations of the Dublin Boulder Clay have

been identified by Skipper and co-workers. As well as at the DPT site all four units have been identified

at the Mater hospital (Fig. 1). South of the River Liffey the third unit (LBrBC) appears to be absent.

Further work is required to define distribution of the various units of the DBC throughout the city.

3.4 Upper brown boulder clay (UBrBC)

The Upper Brown Boulder Clay (UBrBC, the “Brown Dublin Boulder Clay” of Farrell et al., 1995a) was

found to be present in the area of study to a depth of up to 3 m below ground level, see Figure 3a. Farrell LOWER BLACK BOULDER CLAY(LBkBC) LOWER BROWN BOULDER CLAY(LBrBC) UPPER BLACK BOULDER CLAY(UBkBC) UPPER BROWN BOULDER CLAY(UBrBC) LOESS AND RECENT SOIL FLUVIOMARINE GRAVELS, SAND SAND SILTS NORTH SOUTH CARBONIFEROUS LIMESTONE

Figure 2. Simplified schematic of the interpreted stratigraphy of DBC (Skipper et al., 2005).

and co-workers established that the Upper Brown Boulder Clay is a pedogenically altered (soilified)

version of the Upper Black Boulder Clay (see below). In the field it is a “stiff to very stiff yellowish

brown slightly sandy slightly gravelly silt/clay with some cobbles”.

The UBrBC also contains an abundance of palaeo-rootlets, particularly in the upper metre, the UBrBC

does contain a small number of higher permeability sand and gravel lenses which are merely weathered

versions of the small-scale permeable features seen in the Upper Black Boulder Clay (see below).

The Upper Brown Boulder Clay represents a complex pedogenic horizon, which developed during a

period of climate warming and glacial retreat after the deposition of the Upper Black Boulder Clay.

3.5 Upper black boulder clay

The Upper Black Boulder Clay (UBkBC) is typically between 4 m and 12 m thick - a thickness variation

that is principally due to the undulating glaciotectionised contact between it and the underlying Lower

Brown Boulder Clay (Figure. 3b). It is characteristically a "very stiff dark grey to black slightly gravelly

slightly sandy CLAY with some cobbles" and can be described overall as a diamicton (i.e. it has a straight

line particle size distribution curve (see later). Although undisturbed UBkBC appears to be homogeneous

overall, once it is excavated or disturbed its fabric contains a number of discontinuities. Rare, sub-vertical, rough and very tightly closed fissures spaced at 0.5 m to 0.75 m were observed at

some locations, and allowed the machine excavated material to break into peds of similar dimensions.

Figure 3a. UBrBC and UBkBC.

Figure 3b. Undulating contact between UBkBC and LBrBC (c). Rare large granular lens in UBkBC.

In the DPT project, these fissures were observed to be so tightly closed in the in situ material that they

could not be seen in borehole cores, and were only obvious in excavated material and slopes due to the

manner in which the material broke apart. The outer surfaces of these fissures are typically very rough

and gravelly - not polished. The sub-vertical fissures are aligned roughly north-south. Permeable lenses found in the UBkBC have very significant implications for its engineering behaviour

as will be described later. These are generally less than 1.5 m wide and 250 mm thick. Rarely, larger

lenses are found. An example of one up to 3 m wide and up to 1.5 m thick, exposed in the DPT project,

is shown on Figure 3c. Particle size distributions for the lenses range from coarse sand to medium gravel

and are usually crudely sorted - often being coarser in the centre and finer at the periphery. Generally,

although they seep water after excavation, they are

self-draining within a few hours, suggesting relatively

poor interconnectivity. Horizontal cobble lines of striated, faceted cobbles are frequently seen, which persist laterally for

tens of metres before terminating abruptly. These are associated with shear planes within the lodgement

till. The general fabric, cobbles, sub-horizontal cobble lines and sub horizontal discontinuities within

the UBkBC indicate that in addition to being a diamicton in the strictest sense (i.e. is has a generally

straight line grading curve) it may be defined as a lodgement till. This means that it is made of glacially

ground-up bedrock, and was “lodged” at the base of a glacier during a period of glacial advance. The permeable lenses are therefore interpreted as subglacial channel deposits (Brodzikowski and Van

Loon, 1987), and are interpreted as having been deposited by small subglacial channels that were formed,

like the lodgement till, at the glacier’s base. The rare large open fissures and large sub vertical discontinuities are interpreted as surficial crevasses

(Brodzikowski and Van Loon, 1987) that developed in response to stresses within the body and sub-glacial

deforming layer of the glacier.

3.6 Lower brown boulder clay (LBrBC)

At the Dublin Port Tunnel site, the Lower Brown DBC (LBrBC) exists as a 5 m to 9 m thick hard, brown,

silty clay, with gravel, cobbles and boulders, see Figure 3b. It has previously been called the “sandy

boulder clay” as it is similar to but siltier than the UBkBC above. The unit contains more frequent, larger

and more complex silt/gravel lenses and cobble lines than the UBkBC. At the DPT site a continuous

2 m thick layer of silty sand/fine gravel exists within the

unit at 10 m to 16 m depth. Lensoid rafts of

the LBrBC up to 15 m long were found within the UBkBC, close to its true boundary with the LBrBC.

These rafts were slabs of basal material that were detached, transported, redeposited recompact within

the new lodgment till by the glacier. Skipper et al. (2005) sub-divide the LBrBC into three sub units. The upper and lower units are

characterized as diamictons, which have many of the same features as the UBkBC, and are also therefore

interpreted as lodgment tills. The most variable middle unit can be interpreted as a combination of sheet

flood sediments, deposited on a fluvial plain, and laminated sediments deposited in a lake or shallow

estuarine environment. It seems these deposits were laid down during a period of climate warming.

3.7 Lower black boulder clay (LBkBC)

The Lower Black Boulder Clay (LBkBC) has been seen to reach a maximum thickness of approximately

4 m. This unit consists of very stiff dark grey to black slightly sandy slightly gravelly silt/clay with

some to many cobbles to boulders. The principle difference between the UBkBC and LBkBC is that the

lower is generally blacker and more plastic when wetted in hand specimen than the upper. In addition,

the cobble and boulder clasts in the LBkBC are noticeably very angular in the lowermost metre (in the

closest contact with the Carboniferous Limestone). Again, the LBkBC is interpreted as a lodgment till. Occasionally rafts of limestone can be found

within the LBkBC. These are of importance in ground investigations and design as they can be mistaken

for intact bedrock ("false" rockhead). The observed phenomenon can only be explained by glaciotectonic

transport (similar to that seen in the contact between the UBkBC and the LBrBC) of a raft of limestone,

followed by collapse and partial compaction during melting and following glaciations.

3.8 Source of material

In the UBkBC clasts consist of up to 90% Carboniferous Limestone, the remainder consisting of vein

calcite, quartz and rare "exotic" clasts such as Old Red Sandstone or flint. These "exotic" clasts probably

have their origin in Northern Ireland and confirm that the glacier originated in the north and was moving

in a southerly direction. Clasts from the LBrBC were particularly diverse in provenance and included Old Red Sandstone,

schists, quartzites, vein quartz, flint and igneous rocks including a number of granites. The siltiness,

brown colour, and clasts that are found in the LBrBC suggest a different glacial origin from the UBkBC.

The presence of shell material and clasts from the north east of Northern Ireland and Scotland strongly

suggests that this material was deposited by a glacier with a possible Irish Sea origin. In the LBkBC, the cobble and boulder clasts are noticeably very angular in the lowermost metre (in

the closest contact with the Carboniferous Limestone), and the provenance of these lowest clasts is more

locally derived than those in the UBkBC.

3.9 Post depositional processes

As detailed above the geological history of the material is complex. Undoubtedly much of the material

was subject to further glacial induced loading and shearing post deposition. This would have resulted

in the development of variable excess pore water pressures.

Furthermore the UBrBC was produced by

weathering of the UBKBC.

3.10 Stress history

This is perhaps the most complex and poorly understood aspect of the DBC. Groundwater conditions

however have been well investigated. Typical piezometer data for the Ballymun Northern Gateway site are

shown on Figure 4. Pressures are hydrostatic corresponding to a ground water table at about 2 m. Occa

sionally lower values due possibly to drainage into granular lenses or to the underlying bedrock are noted. Given the complex geological history, which at the very least involved a high degree of shearing,

it is not of course possible to determine the stress history of DBC directly from the geological history

of the Dublin sites. Certainly it was unlike that of marine or lakebed deposits, which were sedimented,

consolidated and possibly also unloaded under 1D conditions.

Typical examples of the behaviour of DBC in 1D compression are presented on Figure 5 (Lehane and

Simpson, 2000). Data is given in $\log \sigma'_{v}$ (vertical effective stress) against e (void ratio) form. Results

from four standard oedometer tests are shown: rotary cored samples from 5 m and 7.5 m, an open drive

tube samples from 12 m and a reconstituted mix of the same sample from 5 m, excluding the sand and

gravel content and mixed at an initial water content equal to the liquid limit. It is clearly very difficult to establish a "preconsolidation" stress (p'_{c}) or vertical yield stress (σ'_{vy}).

Lehane and Simpson (2000) and Lehane and Faulkner (1998) used a plot of $\log (1 + e)$ against $\log (\sigma'_{v})$

after Butterfield (1979) for these and other tests to give $\sigma'_{vy} = 1300 \pm 300$ kPa with no apparent variation

with depth. 0 40 80 120 Water pressure (kPa) 14 12 10 8 6 4
 2 0 D e p t h (m) Hydrostatic line with watertable at 2 m
 depth UBrBC UBkBC

Figure 4. Groundwater conditions at Ballymun Northern Gateway site.

Figure 5. Behaviour of DBC in 1D compression (from Lehane and Simpson, 2000). The authors have unsuccessfully used the same and other techniques on other DBC samples. The

bilinear plot suggested by Butterfield is not apparent. In the opinion of the authors the concept of a $p'_{c,0}$, as

applied to normal sedimented clay, is not relevant for this material. Any numerical value for $p'_{c,0}$ that may

be obtained is simply an artifact of the way the data is plotted. For tills it seems likely that the most important mechanism resulting in the overconsolidated state of the

material is shearing during glacial advance rather than compression under the weight of glacier ice. This

has been confirmed by workers at Delft University of Technology and the University of Saskatchewan

(e.g. Garneau et al., 2004). They carried out tests on a Dutch glacial till in a lateral stress oedometer

and confirmed that material to have anisotropic horizontal stresses. The higher stress corresponded to

direction of ice advance. Further work on this aspect of the properties of DBC is well warranted.

4 MATERIAL COMPOSITION

4.1 Grain size distribution - DPT site

Particle size distribution curves for DBC for the four formations at the DPT sites are shown on Figure 6.

Perhaps the most uniform material is the UBkBC. Data for this site compare very well with the range

of values for Dublin black boulder clay presented by Orr and Farrell (1996), see Figure 6 and with the

typical values given by Lehane and Simpson (2000), see Table 2. It can be said that the curves are typical

of those of a lodgement till (Hanrahan, 1977). The single available test for the UBrBC confirms that its

composition is more or less identical to the upper black. Composition of both the LBrBC and LBkBC is more variable. For details of the sub-layers within the

LBrBC, the reader is referred to Skipper et al. (2005). In general this layer contains zones of higher silt

and sand content than the UBkBC. Also shown on Figure 6 are some curves for various layers and lenses within the main strata at the DPT

site, but which are typical of those encountered elsewhere. These include lenses of relatively gravelly

material in the UBkBC. These are often composed of distinct gravel/silt units and may fine up or down.

Deltaic sequences of sands, silts and clays are found in the LBrBC. In general the quantity of the various constituents are relatively constant with depth, as can be seen on

Figure 7. Note these data are for the main strata only and do not include the non-regular lenses, which

were discussed above. Based on the average values, quoted on Table 2 and the plot of the distribution

of the various units with depth shown on Figure 7, it can be seen that there is some tendency for the

quantity of clay and silt to increase with depth. Except for the LBkBC boulder clay, the sand content

remains relatively constant with depth and the gravel content reduces with depth.

Particle size (mm)	0	20	40	60	80	100	Percent	age	passing (%)
Lower black	0.0001	0.001	0.01	0.1	1	10	100		
Upper brown	0.0001	0.001	0.01	0.1	1	10	100		
Range from Orr and Farrell (1996)	0.0001	0.001	0.01	0.1	1	10	100		
Clay	0.0001	0.001	0.01	0.1	1	10	100		
Sand	0.0001	0.001	0.01	0.1	1	10	100		
FineMed. Coa.	0.0001	0.001	0.01	0.1	1	10	100		
Coa.Fine	0.0001	0.001	0.01	0.1	1	10	100		
Fine Med.	0.0001	0.001	0.01	0.1	1	10	100		
Med. Gravel	0.0001	0.001	0.01	0.1	1	10	100		
Silt	0.0001	0.001	0.01	0.1	1	10	100		
Upper brown boulder clay	0.0001	0.001	0.01	0.1	1	10	100		
and Lower black boulder clay	0.0001	0.001	0.01	0.1	1	10	100		
Cobbles	0.0001	0.001	0.01	0.1	1	10	100		
Tests from northern cut & cover	0.0001	0.001	0.01	0.1	1	10	100		
Tests from trial trench	0.0001	0.001	0.01	0.1	1	10	100		
Range from Orr and Farrell (1996)	0.0001	0.001	0.01	0.1	1	10	100		
Upper	0.0001	0.001	0.01	0.1	1	10	100		

black boulder clay 0.0001 0.001 0.01 0.1 1 10 100 Particle size (mm) 0 20 40 60 80 100 Percentage passing (%) Deltaic sequence within lower brown boulder clay Lens within the upper black boulder clay Sand/ silt lenses in trial trench Range from Orr and Farrell (1996) Various layers and lenses within main strata Clay Sand FineMed. Coa. Coa.Coa.Fine Fine Med. Med. Gravel Silt Cobbles 0.0001 0.001 0.01 0.1 1 10 100 Particle size (mm) 0 20 40 60 80 100 Percentage passing (%) Unit A Unit B Unit C Unit C (laminated) Silt Lower Brown Boulder Clay ZBHDHD

Figure 6. Particle size distribution curves DPT site. Table 2. Typical properties of DBC (Lehane and Simpson, 2000). Property Units Value Bulk density kN/m³ 21.5 ± 0.5 Global water content % 11 ± 3 Liquid limit % 25 ± 4 Plastic limit % 14 ± 2 Plasticity index % 11 ± 2 Liquidity index -0.2 ± 0.03 Clay fraction % 15 ± 5 Gravel fraction % 30 ± 5 Permeability m/s 1 × 10⁻⁸ to 1 × 10⁻¹¹

4.2 Grain size distribution - other sites

Plots of particle size distribution for the UBrBC and UBkBC at the Ballymun, Mater, Dáil Eireann and

UCD Belfield sites are shown on Figure 8. Again it can be seen that there is a very high degree of

consistency across the whole area with the curves being similar to the range for Dublin black boulder

clay presented by Orr and Farrell (1996). Particle size distribution for the UBrBC and UBkBC are more

or less identical.

4.3 Grain shapes and form of clay fraction

Some scanning electron microscope (SEM) images of the UBrBC, UBkBC, LBrBC and LBkBC are

shown on Figures 9a to 9d respectively. Some comments on the images are as follows:

- Figure 9a - UBrBC: Possible alignment of plates parallel to plane of photograph with face to face

orientation. Voids very small. Many cracks and fissures due to weathering.

- Figure 9b - UBkBC: Mixture of platy and rotund particles with very tight packing. Pore size 1 μ m

or less. Possible evidence of secondary calcite cementation. 0 20 40 60 80 100 Clay content (%) 30 25 20 15 10 5 0 Depth (m) 0 20 40 60 80 100 Silt content (%) 30 25 20 15 10 5 0 0 20 40 60 80 100 Sand content (%) 30 25 20 15 10 5 0 0 20 Gravel content (%) 30 25 20 15 10 5 0 Upper brown Upper black Lower brown Lower black Upper brown -trial trench Lower black - trial trench 40 60 80 100

Figure 7. Particle size distribution with depth - DPT site. 0.0001 0.001 0.01 0.1 1 10 100 Particle size (mm) 0 20 40 60 80 100 Percentage passing (%) UBkBC - Ballymun UBkBC - Mater UBkBC - Dáil UBkBC - Tallaght UBkBC - UCD Belfield UBrBC -Mater UBrBC -Dáil UBrBC -Tallaght Range UBkBC Farrell and Orr(1996) Clay Sand FineMed. Coarse CoarseCoarse Fine Fine Med. Med. Gravel Silt Cobbles

Figure 8. Particle size distribution - other sites.

Figure 9. SEM images of (a - top left hand side) UBrBC, (b - top rhs) UBkBC, (c - bottom lhs) LBrBC and (d -

bottom rhs) LBkBC.

- Figure 9c - LBrBC: Similar to UBkBC with no obvious alignment of particles.
- Figure 9d - LBkBC: Agglomerates of platy like particles. Again very little void space with possibly

some evidence of secondary calcite cementation. In general, the clayey Dublin boulder clay formations comprise a dense packing of well graded

material, leading to a high bulk unit weight, low pore sizes and hence low permeability in the intact

clayey formations. This is consistent with the in situ development of suctions, following stress relief as

reported by Long et al. (2004).

These photographs support the view that the clay fraction is made up of glacially ground-up bedrock,

which was "lodged" at the base of the glacier during a period of glacial advance. It is clear that the

clay fraction is made up of a mixture of plate like and rotund rock flour particles. Earlier studies had

suggested the absence of plates.

4.4 Clay minerals

X-ray diffraction studies were carried out at the National History Museum, London on samples of each

of the four formations from the DPT site. Specimens tested had particle size less than 2 μm . On average

76% of these specimens were composed of clay minerals. Quartz, calcite and traces of amorphous iron

made up the remainder. A plot of the clay mineral composition with depth is shown on Figure 10. The clay

minerals comprise the following: a small fraction of kaolinite (i.e. 4% to 14% of the clay minerals), with

the balance being split between the illite (28%-43% of the clay minerals) and interstratified illite/smectite

(48%-57% of the clay minerals). There were no clear differences between the specimens from each of the

four formations, except that there was some evidence of expandable clays found in the LBkBC specimens

below 25 m depth. The presence of illite/smectite and kaolinite is suggestive of some surficial weathering.

4.5 Organic content

There is no evidence of organic material in the DBC. Loss on ignition values are generally close

to zero.

Depth (m)	Illite	Illite /smectite	Kaolinite
0	28	48	4
20	35	50	15
15	30	45	25
10	32	48	20
5	30	45	25
0	30	45	25

Upper brown boulder clay Upper black boulder clay Lower brown boulder clay Lower black boulder clay Strata boundaries approximate given for guidance only Limestone bedrock

Figure 10. Clay mineral composition - DPT site.

4.6 Specific gravity

Hanrahan (1977) suggests that the specific gravity (G_s) of Irish glacial tills is nearly always within the

range 2.65 to 2.75. Davitt and Bonner (1977) determined average G_s of 2.67 and 2.72 for the UBrBC

and UBkBC respectively at the site of the Stillorgan bypass (near UCD) in southeast Dublin.

5 STATE AND INDEX PARAMETERS

5.1 Water content and degree of saturation

Values of water content versus depth for the DPT (all values from a single borehole - BH1020W),

Ballymun, Mater Hospital, Dáil Eireann, UCD Belfield and Tallaght sites are shown on Figures 11a to

11f respectively. The sites north and south of the River Liffey are shown on separate pages. Note also

that the same scale is used on the x-axis for all plots. Data for the entire DPT northern cut and cover

site is shown on Figure 12. Water content is highest in the UBrBC, sometimes exceeding 20% but on

average is about 13%. For the UBkBC at the sites north of the River Liffey water content is generally

less than 10% but is slightly greater than 10% for the sites south of the river. Values for the LBrBC and

LBkBC are on average about 11.5%. All of these measured values are similar to the range of $11 \pm 3\%$

suggested by Lehané and Simpson (2000) for DBC. Experience from laboratory testing confirms that below a depth of about 2 m (i.e. the location of the

water table), specimens are generally 100% saturated.

5.2 Bulk density

As for the water content, data for all six sites are shown on Figures 11a to 11f. Bulk density (ρ) values

are generally high and are on average 2.3 Mg/m^3 for all 4 units of the DBC. The sites north of the River

Liffey have higher average ρ ($\approx 2.35 \text{ Mg/m}^3$) than those south of the river ($\rho \approx 2.25 \text{ Mg/m}^3$). Overall

these data reflect the homogeneity of the deposit across the city. 1.6 1.8 2 2.2 2.4 2.6 Bulk density (Mg/m^3) 20 16 12 8 4 0 Depth (m) 0 10 Water content (%) 20 16 12 8 4 0 0.2 0.15 0.1 0.05 0 0.5 1 Liquidity index 20 16 12 8 4 0 Depth (m) UBrBC UBkBC LBrBC Silt w L w P w Key: LBrBC UBkBC UBrBC DPT -BH1020W 1.6 1.8 2 2.2 2.4 2.6 1.6 1.8 2 2.2 2.4 2.6 Bulk density(Mg/m^3) 12 10 8 6 4 2 0 Depth (m) 0 10 Moisture content (%) 12 10 8 6 4 2 0 0.2 0.15 0.1 0.05 0 0.5 1 Liquidity index 12 10 8 6 4 2 0 UBrBC UBkBC UBkBC UBrBC w P w L w Key: Ballymun Northern Gateway Bulk density(Mg/m^3) 20 18 16 14 12 10 8 6 4 2 0 Depth (m) Moisture content (%) 20 18 16 14 12 10 8 6 4 2 0 0.2 0.15 0.1 0.05 0 0.5 1 Liquidity index 20 18 16 14 12 10 8 6 4 2 0 UBrBC UBkBC UBkBC UBrBC w P w L w Key: Mater Hospital LBrBC LBkBC 20 30 20 30 40 40 0 10 20 30 40

Figure 11a to 11c. Bulk density, water content and liquidity index for (a) DPT - BH1020W, (b) Ballymun and

(c) Mater Hospital (Sites north of the River Liffey). 1.6 1.8 2 2.2 2.4 2.6 Bulk density (Mg/m^3) 14 12 10 8 6 4 2 0 Depth (m) 0 Moisture content (%) 14 12 10 8 6 4 2 0 0.2 0.15 0.1 0.05 0 0.5 1 Liquidity index 14 12 10 8 6 4 2 0 UBrBC UBkBC UBkBC UBrBC w P w L w Key: Made ground Dáil Eireann Bulk density (Mg/m^3) 8 7 6 5 4 3 2 1 0 Depth (m) Moisture content (%) 8 7 6 5 4 3 2 1 0 Liquidity index 8 7 6 5 4 3 2 1 0 UBrBC UBkBC UBkBC UBrBC w P w L w Key: Made ground UCD -Belfield- Life Sciences Bulk density(Mg/m^3) 18 16 14 12 10 8 6 4 2 0 Depth (m) Moisture content (%) 18 16 14 12 10 8 6 4 2 0 Liquidity index 18 16 14 12 10 8 6 4 2 0 UBrBC UBkBC UBkBC UBrBC w P w L w Key: Town Centre Tallaght 10 20 30 40 1.6 1.8 2 2.2 2.4 2.6 0 0.2 0.15 0.1 0.05 0 0.5 110 20 30 40 1.6 1.8 2 2.2 2.4 2.6 0 0.2 0.15 0.1 0.05 0 0.5 110 20 30 40

Figure 11d to 11f. Bulk density, water content and liquidity index for (d) Dáil Eireann (e) UCD, Belfield and

(f) Tallaght (Sites south of the River Liffey). 0 Water content (%) 30 25 20 15 10 5 0

D

e p

t h

(m

) Upper brown Upper black Lower brown Lower black Upper
brown - trial trench Upper black - trial trench Liquid
limit (%) 30 25 20 15 10 5 0 Plastic limit (%) 30 25 20 15
10 5 0 clayey lens UBrBC UBkBC LBrBC LBkBC UBrBC UBkBC
LBrBC LBkBC 10 20 30 40 0 10 20 30 40 0 10 20 30 40

Figure 12. Water content, liquid and plastic limit for DPT
northern cut and cover site. Data presented here seem to be
higher than the typical values of $2.19 \pm 0.05 \text{ Mg/m}^3$
suggested by

Lehane and Simpson (2000). Hanrahan (1977) collates w and p
data for a wide range of Irish glacial

tills and finds an approximate linear relationship with p
decreasing from about 2.3 Mg/m^3 at w of 10%

to about 1.8 Mg/m^3 at w of 25%. Hanrahan's data are
consistent with findings here. Hanrahan (1977)

also suggests these high p values are due to the simple
structure (particle arrangement) of the tills.

5.3 Atterberg limits

Liquid (w_L) and plastic (w_P) limit values are plotted
against depth for the same sites on Figures 11 and

12. For all sites both w_L and w_P are relatively constant
with depth and show average values of about

29% and 15% respectively. At the Tallaght site, w_L values
are slightly lower and average about 25%.

Lehane and Simpson (2000) suggest that w_L and w_P are in
the range $25 \pm 4\%$ and $14 \pm 2\%$ respectively

for DBC, consistent with the data presented here.

These same data are plotted on the "A" line plasticity
chart on Figure 13. Again it can be seen that the

deposit is highly uniform across the city. Generally all of
the data for the UBkBC falls in the zone of "clay

of low plasticity" with average plasticity index (I_p) of
about 13%. Values for the UBrBC occasionally

fall in the “clay of intermediate plasticity” zone and I_p is on average close to 14%. Hanrahan (1977) finds a similar linear relationship between w_L and I_p for a wide variety of Irish tills

with I_p in the range 2% to 22%.

5.4 Liquidity index

Liquidity index [$LI = (w - w_P)/I_p$] can sometimes be a very useful parameter for distinguishing between

different strata (e.g. as demonstrated by Hight et al, 2003 for London clay). LI values for the six sites

considered above are shown on Figures 11a to 11f and for the whole of the DPT northern cut and cover

site on Figure 14. For the UBkBC values are usually constant with depth and fall in the range -0.4 to -0.5. For the

UBrBC LI tends to be closer to zero but also relatively constant with depth. These slightly lower values

reflect the weathering processes subjected to this unit. Data for the LBrBC and LBkBC are more scattered

Depth (m)	Upper brown	Upper black	Lower brown	Lower black
0	0.27	0.43	0.17	0.66
10	0.27	0.43	0.17	0.66
20	0.27	0.43	0.17	0.66
30	0.27	0.43	0.17	0.66
40	0.27	0.43	0.17	0.66
50	0.27	0.43	0.17	0.66
60	0.27	0.43	0.17	0.66

Average values:
 Upper brown = 0.27 Upper black = 0.43 Lower brown = 0.17
 Lower black = 0.66 UBrBC UBkBC LBrBC LBkBC

Figure 13. Plasticity chart.

Figure 14. Liquidity index for DPT northern cut and cover site.

Figure 15. Block sampling at DPT project.

but on average are also less than zero. The LBkBC has on average the lowest values being about -0.66,

consistent with this material having been subjected to the most intense shearing. All of these negative

values reflect the highly “overconsolidated” state of the material. Hanrahan (1977) collates I L values for several Irish tills and finds that Dublin till has average value of

about -0.5 as found here. He also points out that the Dublin tills are particularly noteworthy for having

I L below zero.

6 SAMPLING DISTURBANCE

6.1 General

Prior to any discussion on the engineering properties of the DBC it is necessary first to consider the issue

of sampling disturbance. Prior to the mid 1990’s samples were generally recovered by open drive U100

(“U4”) sampling according to British Standard BS5930. Thin walled push sampling proved impossible.

Because of the consistency of the material and the high stone content U100 sampling often failed. Even

when these samples were recovered they could be observed visually to be highly disturbed. Since that

time use has been made of block sampling and rotary coring as follows.

6.2 Block sampling

Undisturbed block samples were recovered from the DPT project, as shown on Figure 15. The block

samples were taken using 300 mm or 350 mm cubical, thin walled steel samplers, with a 20 ° or 45 °

outside angle cutting edge and 9 mm thick sidewalls, respectively. These sampler dimensions give an

area ratio (AR) of 10% to 12% [$AR = (D_e^2 - D_c^2) / D_c^2$], where D_e and D_c are the external and internal

dimensions of the cutting shoe]. Block samples were retrieved with minimum disturbance by steadily pushing the open ended sampler

into the ground, cutting edge first, using the leverage of

a 25 tonne excavator acting through a wood

cushion. Once the box was full of material, the top was waxed to provide a seal, the soil around and below

the sampler was excavated away and the base of the sample was waxed and protected for transportation.

6.3 "Geobore S" rotary coring

Current state of the art in Dublin is to recover high quality cored samples of DBC using "Geobore S"

rotary coring. This is a wire-line, triple tube rotary coring technique with polymer flush to optimise

sample recovery. The wire-line assembly allows the sample to be retrieved immediately after coring

and minimises the duration the sample is in contact with the flushing medium. The drill bit is typically

102 mm ID and is equipped with face discharge, meaning the flushing agent is never in direct contact

with the soil. The innermost tube is the plastic core liner. Drilling is carried out in a borehole of 140 mm

to 150 mm in diameter.

Typically drilling progresses at about 100 bar drill fluid pressure and 800 revolutions per minute

(rpm) of the core barrel. Cores runs are usually 1.5 m. In general, 95% core recovery is achieved with

experienced drilling crews. Cores are logged immediately on recovery and selected lengths are chosen

	0	40	80	120	160	200
Initial effective stress, u_r (kPa)	24	20	16	12	8	4
Depth (m)	1996 rotary cores	2000 rotary cores	2002/2003 rotary cores	2002 blocks	σ_v	0 2 3 4 $\Delta V/V$
(%)	24	20	16	12	8	4
Depth (m)	0	0.04	0.08	0.12	0.16	
$\Delta e/e$	0	24	20	16	12	8
Depth (m)	0	0.04	0.08	0.12	0.16	

Very good to excellent Good to fair Sample quality criteria for clays with OCR 2-4 after Lunne et al.(1997) σ_{ps} Sample quality criteria for clays with OCR 1.5-2 after Kleven et al.(1986) Very good Acceptable Average values of: u_r (kPa) / $\Delta V/V$ (%) and $\Delta e/e$

	2002 / 2003 rotary cores	2000 rotary cores	1996 rotary cores	2002 blocks
Average values of: u_r (kPa) / $\Delta V/V$ (%) and $\Delta e/e$	24.2 / 0.90 / 0.060	2.49/0.112	45.0 / 0.93 / 0.042	25.4 / 0.411 / 0.019

Poor

Figure 16. Sample quality assessment DPT site.

for future laboratory testing. These sections are sealed with cling film and wax prior to transportation

to the laboratory. This technique was used at the DPT site in 2002/2003 and at the Ballymun, Mater and Dáil Eire

ann sites. In the case of the earlier 1996 and 2000 ground investigation at the DPT site a mixture of

conventional double and triple tube core barrels were used.

6.4 Assessment of sample quality for DPT site

On Figure 16 and on Table 3, three methods have been used to assess sample quality at the DPT site

following the work of Lunne et al. (1997) and others. The first plot shows the initial effective stress

(or suction) in the samples, u_r , measured in a triaxial cell by applying an initial cell pressure to the

samples under undrained conditions, and recording the equilibrium pore pressure. Also plotted are;

(i) the normalised volume change, and (ii) the normalised void ratio change – both measured during

consolidation to the in-situ stresses. Lunne and co-workers advocate the latter as a better quantification

of sample quality – as it measures pore volume change with respect to the initial pore volume, rather

than to the total soil volume. For the DPT site, it appears that the residual effective stresses (u_r) are low and only come close to the

in-situ vertical effective stress or Ladd and Lambe's (1963) "perfect sampling" stress near the top of the

sequence. The theoretical "perfect sampling stress" (σ'_{ps}) can be expressed as: $A u$ is the pore pressure co-efficient relevant to a reduction in field deviator stress, which is assumed

to = 0.33 for this very stiff lodgement till. K_0 was assumed = 1.5. If $K_0 = 1$, then $\sigma'_{ps} = \sigma'_{v0}$. These low

values could be due to the testing technique used, as filter paper was attached to the samples to accelerate

consolidation. In addition there was a full size filter stone at the top of the specimen to allow saturation

and back pressure application and to aid consolidation. Both of these details would tend to allow air into

the system and thus reduce the measured u_r . For guidance, the sample quality criteria of Kleven et al. (1986) and Lunne et al. (1997) are shown

on the σ_v/σ'_v and σ_e/e_0 plots respectively, although they were intended for use with moderately Table 3. Sample quality assessment for DPT site. Avg. u_r (kPa) Avg. σ_v/σ'_v (%) Avg. σ_e/e_0

	1996 cores	2000 cores	n/a
Avg. σ_v/σ'_v (%)	45.0	0.93	0.042
Avg. σ_e/e_0	2.49	0.112	2002/2003 cores
	24.2	0.9	0.06
	2002 blocks	25.4	0.41
	0.019	0	40 80 120 160 200
Initial effective stress, u_r (suction) (kPa)	14	12	10 8 6 4 2 0
Depth (m)	NMTL - UU UCD - UU NMTL - CAUC		

“Perfect” sampling stress Ladd and Lambe (1963) assuming $K_0 = 1.0$

Figure 17. Assessment of sample quality Ballymun site.

overconsolidated marine clays. All the block samples fall in the “very good to excellent” category.

For the blocks the average value of σ_v/σ'_v is 0.4 (Table 3).

The 1996 and 2002/2003 cores are also generally of high quality, as most of the data points fall in

the “good to fair” category, with average σ_v/σ'_v of about 0.92. There is more scatter in the data for the

1996 cores and the 2000 cores are generally poorer than the other two sets. The increase in quality with

time possibly reflects improved experience in operating this technique in DBC and also a switch from

air mist to polymer gel as a flushing medium.

6.5 Assessment of sample quality Ballymun site

Similar data is available for the Ballymun site for work

done in early summer 2003. Initial sample

suctions are compared to the idealised “perfect” sampling stress on Figure 17. It can be seen that the

values are relatively high and close to the “perfect” sample stress, confirming the high quality of the

specimens. Suction values also increase with depth and the trend is for little deviation from the “perfect” sampling

line, indicating that sample quality is consistently high. Measured values are generally higher than for

the DPT site. This is likely to be due to the absence of filter drains on the specimens in this case. For the three anisotropically consolidated undrained triaxial (CAUC) test results available, average

V/V_0 is 0.68%. According to Kleven et al. (1986), if V/V_0 is less than 1.5%, the sample quality is

“very good to excellent”. This again confirms the high quality of the samples.

6.6 Conclusions on sampling disturbance

From the point of view of practising engineers these results confirm that triple tube coring and block

sampling both provide high quality samples from which the results of the strength and stiffness tests

are representative of the in situ conditions. As will be shown later U100 open drive sampling is not an

acceptable alternative even if reasonable samples are retrieved.

7 ENGINEERING PROPERTIES

7.1 Apparent preconsolidation stress and OCR

As has been discussed in Section 3.10, in the opinion of the authors the concept of a p'_c , as applied

to normal sedimented clay, is not relevant for this material. Any numerical value for p'_c that may be

obtained is simply an artifact of the way the data is

plotted. Undoubtedly DBC was subjected to intense

shearing and compressive stresses in the past. Further work is required to determine the magnitude of

these stresses as will be discussed in the following section.

7.2 In situ horizontal stress

As many of the more recent developments in Dublin involve two to three levels of basement car parking,

engineers regularly face the problem of attempting to estimate the coefficient of horizontal effective

stress (K_0). Several researchers have attempted to measure K_0 in glacial tills using in situ devices. For

example Handy et al. (1982) obtained $K_0 = 1.3$ for a till in Iowa using a stepped blade dilatometer.

Al-Shaikh-Ali et al. (1981) used hydraulic fracture tests and in situ packer tests on a till in Cheshire UK

and obtained K_0 in the range 0.7 to 1.0. Lutenecker (1990) and Powell and Butcher (2003) used various

techniques, including dilatometers, spade cells and pressuremeters to determine K_0 for tills in Iowa and

Cowden, UK respectively. They found similar results in that K_0 was reported to have high values, of the

order of 3 to 4, for the top 4 m or so and then to decrease to values close to 1.0. Based on this published data engineers in Dublin have to date assumed K_0 is in the range 1.0 to 1.5

for DBC. At the DPT site attempts were made to measure K_0 in the field using high pressure dilatometer tests

and in the laboratory using suction measurements and special triaxial tests. The high pressure dilatometer

(pressuremeter) tests were carried out using the Cambridge Insitu device as described by Mair and Wood

(1987). Estimates of K_0 were obtained using the Marsland and Randolph (1977) analysis as modified

by Hawkins et al. (1990) and these are plotted on Figure 18. Values show considerable scatter and range

Figure 18. Coefficient of in situ horizontal stress (K_0) - DPT site.

between 0.2 and 2.5 with an average of about 1.5 and show a tendency for a decrease with depth as would

be expected. As pointed out by Powell and Butcher (2003) estimates of in situ stress from pressuremeter

tests in tills are difficult due particularly to the stony nature of the material. The values measured appear to

be relatively high and it may be appropriate to correct them for finite membrane length effects according

to the procedure developed by Yu (2004). Some researchers have determined K_0 from sample suction measurements using the filter paper

technique. Some tests were attempted for both the DPT and Ballymun sites. Due to the rough nature of

the core surface, the filter paper had to be placed on the flat ends. These ends were cut using a heavy-duty

rotary saw (concrete cutter). It is likely that any residual suction close to the ends was removed during

this cutting process, thus giving low values. Special triaxial tests were carried out on rotary cored specimens from the DPT and Ballymun sites.

The samples were loaded axially at slow rates and the horizontal stress was increased in order to maintain

horizontal strain close to zero. Horizontal strain was measured by a local (Hall effect) gauge. Three series

of tests were carried out and the results are summarised on Figure 18. In the first the specimens were

saturated and then subjected to the test without any prior consolidation. K_0 values in the range 0.2 to

0.3 were obtained. It was felt that these low values were due to stress relief and sample disturbance.

In order to attempt to overcome these effects in the second series of these the specimens were initially

isotropically consolidated to the in situ vertical effective stress. For these tests K_0 was estimated to be

in the range 0.5 to 0.64. In the third series of tests the specimens were consolidated anisotropically to a

K_0 of 0.6. K_0 was subsequently measured to be in the range 0.45 to 0.53. In these tests it is not clear whether a measurement is being made of the stress condition required

to maintain zero lateral strain on a disturbed sample or the in situ K_0 value. In the absence of further

data from in situ testing (e.g. hydraulic fracture) and lab testing (e.g. lateral stress oedometer) it is

recommended that for DBC designers use K_0 values in the range 1.0 to 1.5 and assess the sensitivity of

the output to the assumptions.

7.3 Stiffness - G_{max}

Menkiti et al. (2004) describe how various techniques were used at the DPT site in order to provide G_{max}

input for finite element analyses of the various deep excavations. These included:

- cross-hole seismic,
- seismic refraction,
- multi-channel analysis of surface waves (MASW),
- shear wave velocity measurements using bender elements on triaxial specimens,
- measurements from Hall effect gauges mounted on triaxial samples,
- values measured using unload/reload loops in the high-pressure dilatometer tests. Based on comparisons with the sophisticated cross-hole technique, the relatively simple MASW

approach was shown to give reliable estimates of G_{max} . For the DPT site G_{max} increases rapidly with

depth from about 250 MPa in the UBrBC to about 1000 MPa at a depth of 8 m (in the UBkBC) and more

slowly below this level to values between 1000 MPa to 1500 MPa in the LBkBC. Further evidence of the reliability and accuracy of the MASW technique is provided by Donohue

et al. (2003) and Donohue (2005) for work at the St. James hospital site. Agreement between cross-hole

and MASW data is excellent. Data from the lab tests suffered due to difficulties inserting the benders

into the stony till. High pressure dilatometer tests gave values at relatively high strains. Subsequent back analyses of an unsupported cut trial excavation (Menkiti et al., 2004) and the lateral

movements of the diaphragm walls supporting shaft WA2 (Cabarkapa et al., 2003), both at the DPT site

have confirmed the MASW G_{max} values to be reliable. Shear wave velocity (V_s) measurements for various DBC sites obtained using the MASW technique

are shown on Figure 19 (Donohue et al., 2003, Donohue, 2005). It can be seen that, except perhaps

for the St. James hospital site, the values are very similar suggesting the material is relatively uniform

across the city.

The practical implication of this work is that the relatively inexpensive MASW technique can be

reliably used for determining in situ shear wave velocity (and hence G_{max}) in DBC. Figure 19 also shows data for London clay (Hight et al., 2003) and Cowden till from the east coast of

the UK (Powell and Butcher, 2003) respectively. DBC stiffness values are extremely high when compared

to other stiff clays. It is about six to eight times stiffer than London clay and about five times stiffer than

Cowden till. 0 200 400 600 800 1000 1200 V s (m/s) 20 16
 12 8 4 0 D e p t h (m) DPT - WA2 Iveagh James I James II
 Tallaght DPT - N C&C Mater Hos. Typical London clay Hight
 et al. (2003) Typical Cowden Till Powell and Butcher (2003)

Figure 19. V s from MASW technique for various sites. 1 10
 100 1000 10000 Effective axial stress (kPa) 0.1 0.15 0.2
 0.25 0.3 V o i d r a t i o , e 5 m core 7.5 m core Data
 from Faulkner (1998) and Lehane and Simpson (2000) 0 2000
 4000 6000 8000 10000 Effective axial stress (kPa) 0 50 100
 150 200 250 M o d u l u s , M (M P a) Janbu (1985) model
 for sand or silt $M = m(\sigma' - \sigma'_a)$ 0.5 with $m = 200$

Figure 20. Typical 1D oedometer test results (Data from
 Lehane and Simpson, 2000, Faulkner, 1998).

7.4 Stiffness - 1D compression - constrained modulus M

Typical oedometer tests for DBC are shown on Figure 5 and
 also on Figure 20 (Lehane and Simpson,

2000 and Faulkner, 1998). This later Figure shows the 1D
 compression behaviour of a rotary cored

sample of DBC in both conventional log stress ($\sigma' - v$)
 against void ratio (e) form and also in $\sigma' - v$ versus

constrained modulus M ($M = 1/m - v$) mode. These curves show
 a gradual increase in M with applied stress and suggest the
 material behaves much

more like sand or silt than clay (Janbu, 1985). For a
 sand/silt, Janbu (1985) suggested:

These plots imply that the material has a modulus number
 (m) of about 200 which is typical for a

natural silt/sand with water content of about 10%. These M
 values are consistent with typical m - v values

of $3.7 \times 10^{-3} \text{ m}^2/\text{MN}$ to $37 \times 10^{-3} \text{ m}^2/\text{MN}$ reported by
 Hanrahan (1977) for Irish tills in the stress range

27 kPa to 54 kPa. Hanrahan suggested this range is
 relatively narrow because of the simple structure or

particle arrangement of these material and the values are
 consistent with apparently negligible settlement

observed in buildings in Dublin even under high foundation pressures.

7.5 Compressibility coefficient (C_c) and swelling coefficient (C_s)

With due regard to the comments made above on the applicability of conventional 1D consolidation

theory to DBC, the curves presented on Figures 5 and 20 suggest the material has compression index

(C_c) of about 0.07. This is equivalent to critical state λ values of about 0.04. Similarly the swelling index

(C_s) is about 0.012 (equivalent to κ of about 0.005).

7.6 Compressibility of reconstituted DBC

Lehane and Simpson (2000) have studied the behaviour of reconstituted DBC. Some data for material

reconstituted at water content equal to the liquid limit is shown on Figure 5. In this case the sand and

gravel fraction was removed, resulting in an inferred higher initial void ratio. For DBC the water content

of the stone fraction alone was found to be negligible, indicating that all the soil moisture is retained in

the soil matrix. As a result of the gravel fraction the intact soils have lower λ (0.03 ± 0.005 compared to

0.04) and κ (0.004 ± 0.001 compared to 0.008) values than the reconstituted material. Gens and Hight (1979) observed similar features in another glacial till and concluded that the water

content of the matrix material was a better indicator of mass strength and stiffness.

7.7 Relationship between stiffness and strain - CAUC triaxial tests

Following measurement of the initial effective stress in the triaxial tests as described above, practice at

UCD is to then saturate the test specimens and consolidate them in two stages. In stage 1, the samples

are isotropically consolidated to half of the in-situ vertical effective stress (i.e. to $0.5\sigma'_{v0}$, where σ'_{v0} is the

in-situ vertical effective stress). In stage 2, the samples are anisotropically consolidated to a stress state

$(\sigma'_{v0}, \sigma'_{h0})$, where σ'_{h0} is the in-situ horizontal effective stress, based on the assumed K_0 value. This final

stress state is maintained until the samples stabilise (i.e. rate of volume change $<0.001\%$ /per minute).

Thereafter, the samples are sheared undrained in compression (CAUC) or extension (CAUE) at a strain

rate of 4.5% per day. Secant shear modulus versus strain plots for CAUC tests at the Ballymun and Dáil Eireann sites

are shown on Figure 21. Tests were carried out on GeoBore-S rotary cores and strains were measured

locally using sample mounted (Hall effect) gauges, see inset Figure 21. Results from tests at UCD and a

commercial laboratory are shown. Similar results for the DPT site are reported by Menkiti et al. (2004).

From the plots it can be observed that the material (UBkBC in this case) exhibits strong non-linearity of

stiffness and that there is good agreement between the triaxial test results and the small strain stiffness

(G_{max}) derived from the in situ MASW testing. Occasionally the resolution of the Hall effect gauges can

be poor and they need to be carefully calibrated before each test. From the point of view of practising engineers, the implication of this latter point is that a combination

of good quality triaxial tests on rotary cored samples and the relatively inexpensive in situ MASW

technique can give the full stiffness-strain relationship. Data for the DPT northern cut and cover site is shown on Figure 22. In this case normalized secant

shear modulus $[G/p' \text{ where } p' = 0.33(\sigma' v + 2\sigma' h)]$ at 0.001%, 0.01% and 0.1% strain is plotted against

depth. Once again it can be seen that:

- stiffness values are extremely high,
- stiffness is highly non-linear,
- it increases with depth,
- stiffness in triaxial compression and extension are more or less the same. In addition there is no significant difference between the block samples and the 2002/2003 rotary

cored samples. At present in Ireland, practicing engineers usually adopt a single “operational” value of

$E_u = 100 \text{ MPa}$ (or $G_u = 33.3 \text{ MPa}$) for design in DBC and in other glacial tills. This was derived from

field observations by Farrell et al. (1995b). From the Ballymun and Dáil Eireann tests, presented on

Fig. 21, this value coincides with strains of the order of 0.01% to 0.1%, which seem reasonable for the

normal level of strain encountered in engineering works.
 $1E-005 \quad 0.0001 \quad 0.001 \quad 0.01 \quad 0.1 \quad 1 \quad 10$ Shear strain (%)
 0 200 400 600 UCD - 4 m UCD - 5 m Dáil Eireann G_{max} from MASW
 at Iveagh site $1E-005 \quad 0.0001 \quad 0.001 \quad 0.01 \quad 0.1 \quad 1 \quad 10 \quad 100$ Shear
 strain (%) 0 200 400 600 S e c a n t s h e a r m o d u l u
 s (M P a) Com. lab - 3.5 m Com. lab - 3.9 m UCD - 4.7 m G
 $_{max}$ from MASW at DPT north c&c Ballymun $G_u \approx 33 \text{ MPa}$

Figure 21. Secant shear modulus versus strain Ballymun and Dáil Eireann sites. $0 \quad 1000 \quad 2000 \quad 3000 \quad 4000 \quad G \quad 0.001\% / p' \quad 0$
 (und.) 24 20 16 12 8 4 0 D e p t h (m) Upper brown
 compression Upper brown extension Upper black compression
 Upper black extension Lower brown compression Lower brown
 extension $0 \quad 1000 \quad 2000 \quad 3000 \quad 4000 \quad G \quad 0.01\% / p' \quad 0$ (und.) 24
 20 16 12 8 4 0 $0 \quad 200 \quad 400 \quad 600 \quad 800 \quad 1000 \quad G \quad 0.1\% / p' \quad 0$ (und.)
 24 20 16 12 8 4 0 Note: X-axis scale changed

Figure 22. Normalised shear modulus versus depth - DPT site.

7.8 Permeability

In-situ mass permeability of the DBC is a key factor in

controlling the behaviour in excavations. For the

DPT site use was made of the natural ability of the DBC to stand unsupported at steep slopes (Menkiti

et al., 2004). In that case it was very important to characterise the permeability of the intact cohesive

clay till and the more permeable zones. During the site investigation works at the DPT site, variable head tests through piezometer tips were

carried out and indicated a permeability of the intact cohesive till of about 10^{-9} m/s (Figure 23a). Tests

in purely silty/granular zones give a range of permeability of 10^{-5} - 10^{-6} m/s. Tests with the filter tip in

mixed cohesive and granular zones, give a wide range of values ranging from 10^{-7} to 10^{-9} m/s. It is

likely that the lower bound of this range reflects the permeability of the intact cohesive till, whereas the

upper bound would reflect that of the granular zones. It was recognised that these tests were unreliable due to the low permeability of the material, the

small changes in water level in the piezometer pipe and the accuracy of the measuring dip meter

1	E	0	1	3	1	E	0	1	2	1	E
0	1	1	1	E	0	1	0	1	E	0	0
9	1	E	0	0	8	1	E	0	0	7	1
E	0	0	6	1	E	0	0	5	k	(m/s)	25
20	15	10	5	0	Lab	triaxial	tests	0	25	50	75
100	125	150	Mean	effective	stress, p'	(kPa)	1E-013	1E-012	1E-011	1E-010	1E-009
1E-008	1E-007	1E-006	1E-005	k	(m / s)	Rowe	cell-	UBrBC	Rowe	cell-	UBkBC
Triax	consol.	-	UBrBC	Triax	consol.	-	UBrBC	Triax	consol.	-	UBrBC

D

e p

t h

(m

) Special in situ tests Original GI - granular zones
Original GI - mixed zones Original GI - cohesive zones
UBrBC UBkBC LBrBC LBkBC Strata boundaries approximate given
for guidance only 1 E 0 1 3 1 E 0 1 2 1 E 0 1 1 1 E 0 1 0 1
E 0 0 9 1 E 0 0 8 1 E 0 0 7 1 E 0 0 6 1 E 0 0 5 k (m/s) 25
20 15 10 5 0 Lab triaxial tests 0 25 50 75 100 125 150 Mean
effective stress, p' (kPa) 1E-013 1E-012 1E-011 1E-010
1E-009 1E-008 1E-007 1E-006 1E-005 k (m / s) Rowe cell-
UBrBC Rowe cell- UBkBC Triax consol. - UBrBC Triax consol.

- UBkBC Samples swell from initial in situ stress

Figure 23. Permeability - DPT site.

(± 10 mm). A new in situ testing method was devised (Boylan and Carolan, 2004), in which vibrating

wire piezometer tips were used and the water was delivered to the top via a fine bore tube, therefore

allowing for significant changes in water level over shorter periods of time and thus improved accuracy.

Test results using this equipment are shown on Figure 23a. Results are quite scattered and vary between

10^{-12} m/s and 10^{-8} m/s with a representative mean of about 10^{-9} m/s. The scatter in the results probably

reflects the variable granular fraction in the material. In the laboratory standard triaxial permeability tests were carried out on rotary cored samples, which

were initially consolidated to an estimated mean effective stress, and the results are given on Figure 23b.

The results of these tests show much less scatter and have an average value of about 5×10^{-11} m/s. This

lower range of the data is consistent with the smaller volume of material being tested when compared

to in situ tests. In addition the laboratory test results represent the vertical permeability and the in situ

tests are more influence by the horizontal permeability. There seems to be a trend for a decrease in

permeability with depth especially in the LBrBC. In order to assess the influence of effective stress on the permeability, Rowe cell tests were carried

out on samples initially consolidated to the in situ vertical effective stress and then swelled back in steps

to simulate excavation. Permeability measurements were made after each swelling stage and the results

are plotted against mean effective stress (p') on Figure 23c. Values of the order of 10^{-9} to 10^{-10} m/s

were measured, with little or no dependence on stress level. Measured values for the UBrBC are slightly

higher than for the UBkBC. Typical permeability values back calculated from the consolidation stage of

triaxial tests on the block samples give similar values. It can be concluded from all the above that typical permeability values for the intact cohesive till in

the UBkBC and UBrBC formations are in the range 10^{-9} m/s to 10^{-11} m/s. For design and FE analyses

at the DPT site, a moderately conservative value for the mass permeability of cohesive till of 10^{-9} m/s

was used (Menkiti et al, 2004). A higher value of 10^{-8} m/s was adopted for the UBrBC. The mass

permeability for silty/ granular zones can be taken as 10^{-6} m/s. A similar range of 10^{-11} m/s to 10^{-8} m/s

for the UBkBC was suggested by Lehane and Simpson (2000).

7.9 Anisotropy of permeability

Comparison between in situ and laboratory tests seems to indicate a small degree of anisotropy of

permeability, with vertical permeability about one order of magnitude lower than the horizontal values.

However further work on this issue is clearly required.
 0 50000 100000 150000 200000 250000 Time (secs) 0 20 40 60 M
e a s u r e d p o r e w a t e r p r e s s u r e (k P a)
Base - cell p loading to 50 kPa Mid-plane cell p loading
Base - shearing Mid-plane - shearing DPT - Test UU2

Figure 24. (a) Typical output from mid plane and base probes and (b) section of sample from same series.

7.10 Coefficient of consolidation (c_v)

Due to the small specimen size generally used in the tests a wide variety of results for the coefficient

of consolidation (c_v) have been reported. Hanrahan (1977), Lehane and Simpson (2000) and Brangan

(2006) suggest values may fall in the range $40 \pm 20 \text{ m}^2/\text{year}$. Given the uncertainty in the measurement

of c_v it is recommended that calculations be performed using M and k as an alternative to c_v .

7.11 Laboratory stress-strain behaviour and stress paths

When considering the undrained stress-strain behaviour of DBC an important issue is the measurement

of pore pressure. In tests at UCD, pore pressure is measured both at the mid plane and the base of the

specimen. Typical output for an unconsolidated undrained (UU) test on a 100 mm diameter, 200 mm high

2002/2003 rotary cored sample from the DPT site is shown on Figure 24a. Initially a cell pressure

is applied so as to measure the initial sample suction (u_r) as describe in Section 6.4. The base probe

responds very quickly but the mid plane probe is very slow to respond. After approximately 42 hours

both probes show an identical response. During the subsequent shearing stage, the response from both

probes is essentially the same. Deviators stress-strain, pore pressure-strain and stress path plots [in $s' = (\sigma' v + \sigma' h)/2$, $t' = (\sigma' v - \sigma' h)/2$

space] for various samples for the DPT site are shown on Figure 25. This plot includes tests on both

the UBkBC and UBrBC in compression and extension. A similar set of results for the Ballymun site

(UBkBC in compression only) is shown on Figure 26. In general the test results are very similar and the

following general conclusions can be made: • The materials exhibit a strong dilatant response throughout most of the shearing process as can be seen from all the plots, especially the stress path and pore pressure data. • The material possesses a strong anisotropy of undrained shear strength with s_u in compression being three to four times that in extension. • No similar anisotropy of effective strength parameters is evident. • Material behaviour is

similar over the range of K_0 values (1.0 to 3.0) used in the tests.

7.12 Undrained shear strength from lab testing - definition of s_u

Interpretation of s_u from triaxial tests on DBC is not straightforward given the dilatant nature of the

material and the progressive increase in shearing resistance with strain (Figures 25 and 26). Some

possible techniques for interpreting s_u are as follows:

1. Simple peak deviator stress regardless of strain (conventional approach)
2. Limit strain, e.g. take strength at 2% strain
3. Peak principle stress ratio (σ'_1 / σ'_3)
4. Peak pore pressure
5. $A = 0$ or $u = 0$
6. Reaching Mohr-Coulomb line. 0 8 12 16 Axial strain (%)
0 400 0 400 800 1200

D

e v

i a

t o

r s

t r e

s s

(k

P a

) DPT 7, LBrBC at 9.0 m, $K_0 = 1.0$ DPT9, UBkBC at 5.8 m, $K_0 = 3.0$ DPT2, UBkBC at 4.1 m, $K_0 = 1.5$ 0 8 12 16 Axial strain (%) 0 100 0 100 200 300 400 500

P o

r e

p

r e

s s

u r

e

(k P

a) 0 200 400 600 800 1000 $(\sigma'_1 - \sigma'_3)/2$ (kPa) 200 0
200 400 600 $(\sigma'_1 + \sigma'_3)/2$ (kPa) $\phi' = 44$ deg. $\phi' =$
36 deg. Tests on UBkBC and UBrBC at UCD All samples are
2002/2003 cores. $\phi' = 36$ deg. 4 4

Figure 25. Triaxial test results for DPT site. 0 8 12 16
Axial strain (%) 400 0 400 800 1200

D

e v

i a

t o

r s

t r e

s s

(k

P a

) Com. Lab - 3.5 m Com. Lab - 3.9 m UCD - 4.7 m 0 8 12 16
Axial strain (%) 100 0 100 200 300 400 500

P o

r e

p

r e

s s

u r

e

(k P

a) 0 200 400 600 $(\sigma'_1 - \sigma'_3)/2$ (kPa) 100 0 100 200
300 400 $(\sigma'_1 - \sigma'_3)/2$ (k P a) $\phi' \approx 44$ deg. $\phi' \approx 36$
deg. All sample sare GeoBore-S cores. K_0 assumed = 1.5 4
4

Figure 26. Triaxial test results for Ballymun site.
Hanrahan (1977) suggested that it might be preferable to
adopt criterion 3, although he does not give

any comparative results. Criterion 5 does not always occur.
Criteria 4 and 5 seem to yield conservative

values of s_u . It would seem the best approach would be to
link s_u to some limiting strain value so as

to ensure serviceability limit state is satisfied for
engineering structures. At present the conventional

approach of the simple peak is used in Ireland and this
will be adopted here for comparative purposes.

7.13 Undrained shear strength from lab testing - UU

Traditionally s_u for DBC has been obtained from a
combination of in situ standard penetration tests (SPT)

results and laboratory unconsolidated undrained (UU)
triaxial tests. Although the problems associated

with UU triaxial testing are well understood, they are
frequently used in practice given the time and cost

constrains of carrying out more sophisticated tests. This
is a particular issue for the Dublin tills where

saturation and consolidation of 100 mm diameter specimens
can take up to one week. Results for UU tests on Geobore-S
cores at the DPT (all from BH1020W) and Ballymun sites are

shown on Figures 27a and 27b respectively and all results for these two sites are summarized on Tables

4 and 5. It can be seen that the values are reasonable and are consistent with other test results and the

observed in situ behaviour of the till. In contrast results for the UCD and Tallaght sites (Figures 27e and 27f) show relatively low values of

s_u . These tests were carried out on open drive U100 ("U4") samples. Clearly sample disturbance effects

have significant influence on the tests.

7.14 Undrained shear strength from lab testing - CIUC and CAUC

Undrained shear strength (s_u) measured in anisotropically consolidated and isotropically consolidated

(CAUC or CIUC) for the Ballymun, Mater Hospital and Dáil Eireann sites are shown on Figures 27b,

27c and 27d respectively. Results for the DPT and Ballymun sites are summarized on Tables 4 and 5. Despite some scatter in the data, there is a trend for increasing strength with depth in the UBrBC and

UBkBC from a value of about 80 kPa in the UBrBC to about 500 kPa at the boundary of the UBkBC

and LBrBC clays. Absolute values of CAUC strength are not surprisingly higher than for CIUC tests

(i.e. isotropically consolidated with $K_0 = 1.0$). The slightly lower values for the Dáil Eireann site may

be due to test limitations (range of load cell used). An interesting result for all the material is that s_u values from UU and CIUC tests are very similar.

This result has practical implications in that, once high quality rotary core samples are available, it may

be adequate to carry out cheaper UU rather than more expensive and time consuming CIUC tests. These high strength values are consistent with field observations. Significant mechanical effort is

required to excavate the material and the perceived strength and stiffness increases with depth.

7.15 Undrained shear strength from laboratory testing - CAUE tests

Extension strength (CAUE) data for the DPT site is shown on Figure 25 and all results are summarized on

Table 4. (K_0 was assumed = 1.5). These tests give surprisingly low values, which increase approximately

linearly from about 20 kPa near the surface to 130 kPa at 12 m depth. The specimens failed in necking

with the neck occurring at various locations. An example for a test on a Geobore-S core sample from

the DPT sites is shown on Figure 28. Hight et al. (2003) report a similar mode of failure for extension tests on London clay. Strength

anisotropy ratio $s_u\text{-CAUE} / s_u\text{-CAUC}$ is about 0.25. Other researchers (e.g. Karlsrud et al, 2005) suggest that

this ratio should be of the order of 0.4 to 0.5 for material with an I_p of 10% to 15%. It was observed

that most of the extension test samples failed by forming shallow dipping failure planes, which often

engaged pre-existing discontinuities. It seems likely that the low values are due to the granular nature of

the material. It must be noted that the degree of "anisotropy" depends on how the s_u value is interpreted from the

triaxial test. As can be seen from Figure 25 peak strength in compression occurs at a higher strain than

that in extension. For engineering works, control of deformation may be the dominant design factor and

design strengths must be consistent with tolerable strain. Further study is warranted to investigate the

implied anisotropy of undrained shear strength and the practical implications for engineering works.

7.16 Undrained shear strength from in situ testing - SPT tests

Standard penetration test "N" values for the six sites under consideration are shown on Figures 27a to

27f. All data with equivalent "N" > 100 have been omitted due to probable influence of cobbles. Like

UU tests the problems with SPT testing have been thoroughly discussed in the literature. However, as

the test is performed routinely in ground investigations in Ireland and elsewhere, some results will be

considered here. The observed scatter in the data is considerable and typical, reflecting the large and variable stone

content of the till. At any depth values can typically vary by ± 30 . The data does show that the UBrBC 0 200 400 600 s u (kPa) 20 16 12 8 4 0 D e p t h (m) UU - UCD s u 6 x SPT N Upper brown Upper black Lower brown Silt 0 20 40 60 80 100 0 20 40 60 80 100 SPT N (Blows/300 mm) 14 12 10 8 6 4 2 0 D e p t h (m) UBrBC UBkBC 0 200 400 600 s u UU or CAUC (kPa) 14 12 10 8 6 4 2 0 D e p t h (m) UU - NMTL UU - UCD CAUC - NMTL CAUC - UCD UBrBC, Avg. 18 UBkBC, Avg. 50 s u 6N line UBrBC UBkBC Ballymun Northern Gateway DPT BH1020 SPT N (Blows/300 mm) 20 18 16 14 12 10 8 6 4 2 0 D e p t h (m) UBrBC UBkBC LBrBC LBkBC CAUC - UCD UBrBC, Avg. 22 UBkBC Avg. 64 Mater Hospital 0 200 400 600 s u UU or CAUC (kPa) 20 18 16 14 12 10 8 6 4 2 0 D e p t h (m) s u 6N line UBrBC UBkBC Peak?

Figure 27a to 27c. Standard penetration N and lab s u values for (a) DPT - BH1020W, (b) Ballymun and (c) Mater

Hospital (Sites north of the River Liffey). 0 20 40 60 SPT N (Blows/300 mm) 14 12 10 8 6 4 2 0 D e p t h (m) UBrBC UBkBC 0 200 400 600 s u UU or CAUC (kPa) 14 12 10 8 6 4 2 0 D e p t h (m) CAUC - UCD UBrBC, Avg. 26 UBkBC Avg. 57 s u 6N line UBrBC UBkBC Dáil Eireann SPT N (Blows/300 mm) 18 16 14 12 10 8 6 4 2 0 D e p t h (m) UBrBC UBkBC 0 200 400 600 s u UU or CAUC (kPa) 18 16 14 12 10 8 6 4 2 0 D e p t h (m) UU - Com. Lab UBrBC, Avg. 29 UBkBC Avg. 55 s u 6N line UBrBC UBkBC Tallaght Town Centre SPTN (Blows/300 mm) 8 6 4 2 0 D e p t h (m) UBrBC UBkBC s u UU or CAUC (kPa) 8 6 4 2 0 D e p t h (m) UU - Com. Lab UBrBC, Avg. 27 UBkBC Avg. 71 s u 6N line UBrBC UBkBC UCD Life Sciences 80 100 0 20 40 60 0 200 400 600 80 100 0

20 40 60 80 100

Figure 27d to 27f. Standard penetration N and lab s_u values for (d) Dáil Eireann (e) UCD, Belfield and (f) Tallaght

(Sites south of the River Liffey).

Figure 28. Necking type failure for CAUC test on DBC - DPT site.

Table 4. Average SPT "N" and laboratory s_u values - DPT site. Avg. SPT "N" Avg. CAUC CAUC Avg. CIUC CIUC Avg. UU UU (Blows/300 mm) s_u (kPa) s_u /N avg . s_u (kPa) s_u /N avg s_u (kPa) s_u /N avg

UBrBC 19 84 4.4 n/a - n/a -

UBkBC 53 373 7.0 287 5.4 217 4.1

LBrBC 53 520 9.8 297 5.6 383 7.2

LBkBC 68 n/a - 240 3.5 n/a - Table 5. Average SPT "N" and laboratory s_u values - Ballymun site. Avg. SPT "N" Avg. CAUC CAUC Avg. UU UU (Blows/300 mm) s_u (kPa) s_u /N avg . s_u (kPa) s_u /N avg UBrBC 18 n/a - n/a - UBkBC 50 240.8 4.8 340.2 6.8

has lower strength than the UBkBC. Stroud and Butler (1975) and Stroud (1989) suggests that for glacial

till with average plasticity index of about 12%, the ratio $f_1 = s_u$ /N avg is about 6.0. Farrell and Wall (1990)

report similar f_1 for the UBrBC and UBkBC. In both cases these findings were based on CIUC or UU

triaxial tests. Data for the DPT and Ballymun sites are summarized on Tables 4 and 5 and suggest $f_1 = 6$

may be appropriate for UU and CIUC tests but that a higher value of f_1 applies to CAUC tests. Given the scatter in the test data SPT test results should be used with extreme caution and probably

are useful only as an index test or for reflecting difference across a single site.

7.17 Undrained shear strength from in situ testing - pressuremeter testing

Some in situ s_u data are also available from high pressure dilatometer tests at the DPT site, as shown

on Figure 29a. Data were interpreted according to the limit pressure approach advocated by Powell and

Butcher (2003) for Cowden till ($s_u = \text{limit pressure}/6.18$). This analysis resulted in very high values of

s_u , being on average about 835 kPa for the UBrBC and LBrBC.

These authors show that this approach gives results consistent with large diameter plate tests. Other

available interpretation techniques (see review by Mair and Wood, 1985) often give much higher values

s_u (kPa)	0	400	800	1200	1600
s_u (kPa)	24	20	16	12	8
Depth (m)	0	4	8	12	16

High pressure dilatometer UBrBC UBrBC LBrBC $s_u \approx 5 \times \text{SPT } N$

Figure 29. High pressure dilatometer testing - DPT site (b) CPT testing - Clonee (Faulkner (1988)).

of s_u . This is likely to be due to the fact that most of these techniques were developed for contractant

sedimentary clays or weak rocks rather than the highly dilatant material under consideration here. Further

research is required on the interpretation of s_u from pressuremeter tests in glacial till.

7.18 Undrained shear strength from in situ testing - CPT

CPT (cone penetration tests) tests in DBC are rare due to the high strength and stone content of the

material. CPT operators are often reluctant to attempt tests in the material. A small number of tests have

been recorded, see for example Figure 29b. This test was reported by Faulkner (1998) for a test at Clonee

on the north side of the city, where 2 m of UBrBC overlies UBrBC. The test was suspended at about

4.7 m due to equipment limitations.

7.19 Undrained strength values in till reported by others

The s_u values for the Dublin till are considerably higher than values reported for tills from elsewhere.

For example Powell and Butcher (2003) report s_u values for Cowden till to be in the range 90 kPa to

150 kPa, for high quality pushed samples. Dobie (1989) and Al-Shaikh-Ali et al. (1981) report similar

values for UU tests on U100 specimens from two different sites in Cheshire. Southern England tills have

a lower stone content than those found in Dublin. Slightly lower values of s_u than those encountered in Dublin were reported by Chegini and Trenter

(1996) for UU triaxial tests on U100 samples of Scottish glacial till from Chapelcross, Dumfriesshire.

Mc Kinlay et al. (1974) report lower values of s_u , in the range 100 kPa to 200 kPa for UC, UU laboratory

tests on U100 samples and large diameter plate tests on another Scottish till from Glasgow. Robertson

et al. (1994) report s_u values in the range 50 kPa to 410 kPa for the tills of North East England. In these

cases the lower bound values probably reflect sample disturbance and the lack of correct effective stress

during sample shearing. Lutenecker (1990) reports s_u values of between 100 kPa and 150 kPa for CIUC triaxial tests on Shelby

tube samples of a relatively stone free till from Iowa, USA.

7.20 Effective stress strength parameters

From the stress path plots, for the DPT and Ballymun sites, presented on Figures 25 and 26, it appears: • The materials have a peak effective friction angle (ϕ'), in compression, of about 44° . • Large deformation strength corresponds to ϕ' of about 36° . This similar to the critical state friction angle (ϕ'_{cv}) of $34 \pm 1^\circ$ suggested by Lehane and Faulkner (1998). • Thus the material has a curved rather than linear failure surface, similar to the conclusions of Atkinson and Little (1988) for St. Albans till and Robertson et al. (1994) for

Northumberland till. σ_1 100 200 300 Initial sample stress
(kPa) 30 35 40 45 50 55 60 ϕ 1 (d e g .) Com. lab - UU
UCD - UU Com. lab - CAUC UCD - CAUC

Figure 30. Peak friction angle versus initial sample stress
- Ballymun site.

- This divergence of the stress paths from the linear Mohr-Coulomb envelope at high strains may be

indicative of the development of failure along some
fissures or joints within the specimen rather than

a mass failure of the material, similar to that reported by
Vaughan and Walbancke (1975) for Cow

Green till.

- In triaxial extension ϕ' appears to be about 36° , i.e. similar to the critical state or large strain compression value. This again may be due to failure along some microstructural feature in the material.

- Effective cohesion (c') is close to zero. This is consistent with the small and often rotund clay-sized

fraction, the mode of deformation, involving a high degree
of shearing and lack of evidence of

cementation bonding between the particles. Peak friction
angles (assuming $c' = 0$) have been plotted against either
the initial sample stress (σ_1)

for UU tests or the mean consolidation stress ($p' = 0$) for
CAUC tests for the Ballymun site on Figure 30.

There is a tendency for decreasing friction angle with
increasing effective stress. A similar finding was

made by Menkiti et al. (2004) for the DPT site.

7.21 Sensitivity

Given the mode of formation of DBC it would be expected
that it have very low sensitivity. Gens and Hight

(1979), Atkinson and Sällfors (1991) and others have shown
that the behaviour in shear of reconstituted

glacial till compares well with the behaviour of the intact

till. Due to the high stone content it is difficult

to estimate sensitivity of the Dublin tills in situ using a device such as the field vane. However Atkinson

and Little (1988) report on in situ vane tests in the relatively stone free East Anglian till and found

sensitivity values in the range 3.7 to 5.8, with intact undrained shear strength of the order of 400 kPa. Lehane and Faulkner (1998) report results of triaxial tests on reconstituted DBC. All drained and

undrained stress paths terminate on a straight lines passing through the origin, when p' at failure exceeds

200 kPa. These lines corresponds with $\phi' =$ of $34 \pm 1^\circ$ in both compression and extension. This value is

identical to ϕ'_{cv} and confirms the behaviour of intact DBC in shear compares well with the behaviour of

reconstituted material.

7.22 Large strain (residual) strength

Loughman (1979) studied the residual shear strength of DBC and found that in general the ϕ'_{res} was high

($> 30^\circ$) due to the high silt content of the material, see Figure 31. Loughman (1979) compared his data to that of Vaughan and Walbancke (1975) and found that the

same general trend applied. (Note Vaughan and Walbancke, 1975 suggested the break in the trend line

Figure 31. Residual strength of DBC against I_p (Loughman, 1979).

Figure 32. 33 m high steep slopes in DBC at Howth.

at I_p at about 30% should be treated with caution.) This confirms that, as would be expected from the

mode of formation of the material, it has low sensitivity.

8 ENGINEERING PROBLEMS AND CASE HISTORIES

8.1 Slope stability - stable slopes

There are numerous examples of natural steep slopes in glacial tills in the Dublin area. Hanrahan (1977)

describes near vertical slopes of up to 33 m high in Howth (north of Dublin), Killiney (south) and

Chapelizod (west). There are similar high natural cuts along the River Dodder, which runs through the

south of the city. A view of the cliffs at Howth are shown on Figure 32. Although the cliffs are high

and steep, homes have been developed adjacent to them and have remained intact for over 100 years.

Some local instability can be seen. For example part of the slope beneath the Martello tower, towards the

rear of the photograph, failed in December 2002 following a period of heavy rain and caused the road

underneath to be blocked. Temporary cuts of significant thickness, made for engineering works, can stand unsupported for

relatively long periods, see Figure 33 for example (from Long et al., 2003). Groundwater level, as

Figure 33. Unsupported steep cuts in DBC (a) south side of city, (b) central Dublin. -20.00 0.00 20.00 40.00 60.00 80.00 100.00 24-May-02 01-Sep-02 10-Dec-02 20-Mar-03 28-Jun-03 06-Oct-03 14-Jan-04 23-Apr-04 P o r e p r e s s u r e (k P a) 4 m 8 m 12 m Excavation lifts

Figure 34. Pore water pressure measured at Chainage 130W - DPT site.

recorded by piezometers, was at about 2 m depth in each case. The cut in Figure 33a was made during

the summer time on the south side of the city, and is mostly in the UBrBC. The cut in Figure 33b was

made during the winter near the city center, mostly in UBkBC. No ground water control system was

evident within the excavations and no cracks were observed at the slope crests, for a period of several

months. The authors have seen other near vertical 8 m deep

cuts in central Dublin stand unsupported,

adjacent to heavily trafficked roads, for periods of several months.

Although the material may possess a small c' component and arguably some minor cementation, it is

unlikely that these factors alone could explain its behaviour. The view was taken that the existence of a

c' or cementation could not be relied upon for design. This is because any cementation or bonding could

be fragile, may not be persistently distributed within the till and may be brittle. It is likely then that the explanation of the behaviour of the soil lies in the development of high pore

water suctions (or at least significantly depressed pore water pressures) following stress relief due to

excavation. Positive pressure increases pore water pressure (u). Unloading reduces u , possibly inducing

suctions. Effective stresses and therefore soil strength are thus increased. The tendency of the till to dilate

during shear, induced by the steepness of the cut slope, also contributes to the depression of the pore

pressures/generation of suctions - as illustrated in the triaxial test results shown on Figures 25 and 26. Long et al. (2004) report on suctions measured during works at the DPT site, using specially designed

piezometers. An example of the output is shown on Figure 34. Although the suctions are small they have

strong influence on the slope stability.

Figure 35. Slope instability DPT site (a) Failure of 11 m high trail excavation after 1 to 1.5 months (b) Cracking at

crest of 60° slope at the DPT site. Slope height was 6 m to 6.5 m.

8.2 Slope instability

Failures in DBC slopes are also encountered. Hanrahan

(1977) discussed some rotational failures in a natural 35 ° slope at Glencullen just south of the city. He attributed the failure to internal seepage and stream erosion. During December 2000, the railway line adjacent to the sea cliffs at Killiney (south Dublin) had to be closed on three occasions due to landslides. These were due to surface slips in the upper (weathered) portion of the till which occurred after periods of high rainfall. Some recent instability in the sea cliffs at Howth has also occurred as described above. Menkiti et al. (2004) report on the failure of some sections of the trial cut at the DPT site (Figure 35a). Long et al. (2003) describe some temporary slope failures at Dublin Airport. These failures typically occur after periods of high rainfall, at the location of granular lenses and are characterized by the slip of thin flakey wedges. Such a failure, again from the DPT site, is shown on Figure 35b. The crack in the concrete binding at the slope crest was 3 mm to 8 mm wide with no vertical step across the crack. Weeping lenses of sand and gravel were noted on the face beneath the crack, similar to that in the trial excavation. The conclusion that suctions play a dominant role in the stability of slopes in glacial till is consistent with the observations that gravelly or sandy portions of the slopes fail following rainfall. Clearly the suctions are released quickly in these circumstances resulting in collapse. If monitoring instrumentation (such as inclinometers at the DPT site) is present advance warning of potential slips can be obtained (Menkiti et al, 2004). This is due to the ductile/dilational nature of the material and has significant implications for the use of the observational approach.

8.3 Lightly supported excavations

Although different mechanisms may be involved when compared to cuts and natural slopes the behaviour

of lightly supported excavations in Dublin boulder clay may also reflect the general competence and

inherent strength of the ground and the influence of induced suctions. A summary of available data on

the performance of deep basements in Dublin has been given by Brangan and Long (2001) and Brangan

(2006) and is summarised on Figure 36. Of these 13 case histories, 7 were monitored by researchers from UCD. In all cases the measured

lateral wall movements were very small, typically less than 5 mm, and were close to the accuracy of the

surveying system. The data are plotted as maximum lateral movement (Δh) normalised by excavation depth (H) against

Clough and O'Rourke (1990) system stiffness ($EI/\gamma_w s^4$) on Figure 36. [EI is the wall stiffness, γ_w is

the unit weight of water and s is the average prop spacing.] Dublin data can then be compared with a

worldwide database. The $\Delta h/H$ values are very low and fall well below the line suggested by Clough

and O'Rourke (1990) for cases where the factor of safety against excavation base heave (FOS) is high.

All of the Dublin examples also fall below the average value of 0.14% reported by Long (2001) for 147

worldwide case histories for excavations in stiff soils. A typical example of such a deep basement at the Dáil Eireann site is shown on Figure 37. Here

measured movements of the wall were 3 mm or less and monitoring of the props showed that forces were

negligible. 0 0.1 0.2 0.3 0.4 0.5 0.6 0.1 1 10 100 1000
Clough et al.(1989) system stiffness $EI/\gamma_w s^4$ N o r m . l
a t . w a l l m o v e . ($\Delta h / H$) (%) Dublin case

histories Clough & O'Rourke (1990) for FOS = 3 Avg.147
world case histories, Long (2001)

Figure 36. Behaviour of lightly supported excavations in
DBC.

Figure 37. Deep excavation at Dáil Eireann site. It is
clear that all of the retaining walls have behaved very
well. However it could be also concluded

that current design practice must be very conservative.

8.4 Conventional pad or strip footings

Wall and Farrell (1990) state that foundations in the UBrBC
have been designed for bearing pressures

of 100 kPa to 200 kPa. Allowable bearing pressures (q_{all})
on the UBkBC have varied from 250 kPa

to 500 kPa. In current practice in Dublin the actual value
is often derived from the simple following

equation. A generous factor of safety (FOS) of 2.5 to 3 is
applied.

Figure 38. Trial footing test results (Farrell et al.,
1995b). Farrell et al. (1988, 1995b) and Farrell and Lehane
(1996) report on a full-scale footing trial carried out

at the Tallaght town center site. This footing was 1.5 m
square, of reinforced concrete, was founded on the

surface of the UBkBC and was loaded to a maximum bearing
pressure of 1013 kPa. The measured long

term settlement against applied pressure is shown on Figure
38 and is compared with various predictions

that assumed:

(i) A linear elastic model with a constant drained
vertical Young's modulus (E'_{ν}) = 80 MPa and a Poisson's
ratio (ν) of 0.2.

(ii) A non-linear elastic model with the strain dependent E'_{ν}
similar to that shown on Figure 21. It is evident that
for settlements up to 12.5 mm, when the applied load was
about 250 kPa, the linear elastic

prediction does not differ by more than 1.5 mm from the measured settlement. In this instance the use of a

non-linear stiffness did not enhance predictive performance. The equivalent undrained Young's modulus

corresponding to $E'_{\nu} = 80 \text{ MPa}$ is $E_u = 100 \text{ MPa}$.

8.5 Piled foundations

Square sectioned driven precast concrete piles, with side lengths varying from 250 mm to 350 mm, are

the most popular driven piles in Dublin. These piles are usually driven to a penetration of 2 m to 4 m into

the UBkBC and are deemed acceptable when a required set is achieved. Contractors quote working load

capacities of typically 700 kN and 1000 kN for the 250 mm and 350 mm piles respectively. Design engineers often limit the ultimate shaft resistance (f_s) and the ultimate base resistance (f_b) to

about 100 kPa and 10000 kPa respectively for both driven and bored piles. Research at TCD (Farrell et al., 1998) suggests that higher values can be achieved in practice for

driven piles but that both f_s and f_b are critically dependant on the excess pore pressures present. Jardine (1992) reported on the development of negative pore pressures caused by dilation during the

pushing of jacked piles into the Cowden till. He suggested that this phenomenon accounted for the loss in

driving resistance on re-driving some precast concrete piles at the Tallaght site as reported by Armishaw

and Bunni (1992). These authors felt that the phenomenon was most likely due to creep crushing of

grains in the highly stressed zone around the pile shaft. Farrell et al. (1995b) and Farrell and Lehane (1996) report on some pile load tests on driven precast

concrete piles carried out at Tallaght and at Croke Park just north of the city centre, see Figure 39. Pile

settlements are modest under working loads (always less

than 5 mm). As may be seen on Figure 39 the

load/settlement curves estimated using finite element analysis with a constant E_u of 100 MPa gave an

upper bound to the recorded settlements. A better fit to the measured data is obtained if E_u is assumed

to equal 150 MPa.

Figure 39. Pile load/settlement curves for Tallaght and Croke Park sites (Farrell et al., 1995b).

Figure 40. Soil nailing at Dublin Airport Extension (Pedley, 2000). It is evident, as for the trial footing reported on above, that use of an appropriate linear modulus, at

normal working stress level, is sufficiently accurate for the prediction of settlement. More recently, due to noise and vibration limitations conventional rotary bored or continuous flight

auger (CFA) piles have been used in central Dublin. Lawler (2003) reports on some research on the

behaviour of a CFA pile in DBC. Phillips and Lehane (1996) report on some tests on laterally loaded piles in DBC. They found that the

piles had higher lateral capacity than would be predicted using a conventional design approach. Lateral

pile movement could be predicted reliably using a simple elastic - perfectly plastic soil model.

8.6 Soil nailing

Soil nailing is increasingly being used in many projects in glacial tills in Ireland, particularly to provide

temporary support to steep slopes. Some examples have been for the extension of Dublin Airport (Pedley,

2000) and for the DPT northern cut and cover works (Menkiti et al., 2004 and Milligan et al., 2006).

A plan and cross section through the works at Dublin Airport is shown on Figure 40. Nails were com

prised of 25 mm or 28 mm threaded bars installed in a 114 mm borehole. They were 6 m to 8 m long and

were installed at a constant vertical spacing of 1.5 m. Horizontal spacing varied between 1.25 m and 2 m. Pull out tests demonstrated bond stresses of more than 200 kPa, well in excess of the design ultimate

value. 0 2 8 Distance from end of nail (m) 20 0 20 40 60 A
v e r a g e s h e a r s t r e s s (k P a) End of Lift 1
Soil nailing Lift 2 End of Lift 2 End of Lift 3/4 End of
monitoring period 4 6

Figure 41. Average shear stress induced in Nail 2, Chainage 980E, DPT project (Menkiti and Long, 2006). Little design guidance is available and it is known from the Dublin Airport project and elsewhere that

application of design procedures developed for other materials is conservative. Therefore an opportunity

was taken during a large scale soil nailing works for the Dublin Port tunnel project to instrument some nails

in order to observe their true behaviour (Menkiti and Long, 2006). It was found that the behaviour of nails

was the reverse of that assumed in current design methods. Most load was induced due to drilling and

nailing the lift immediately below the nail rather than due to excavation induced stress relief, see example

on Figure 41. Highest forces were developed in the upper nails, rather than the lower nails as assumed

in design. It would seem that the behaviour the nailed slope is governed more by local effective stress

changes immediately adjacent to the nails than by large-scale development of failed wedges. Despite

this the trial confirmed that the currently used procedures are highly conservative for Dublin glacial till.

9 CONCLUSIONS

There are significant economic benefits in understanding the characteristics of Dublin Boulder Clay

(DBC) as it underlies much of the city. This paper gives geotechnical characteristics of the various

formations comprising the DBC and some of the main findings, which may be of concern to practicing

engineers, are as follows: 1. There are clear differences in the engineering parameters between the four units of DBC as defined by Skipper et al. (2005). 2. The two uppermost units are the most important from the point of view of engineering projects and these seem to be relatively homogenous with depth and with location throughout the city. 3. These can best be distinguished by routine index tests such as particle size distribution, water content and Atterberg limits. 4. Localised granular lenses can be encountered. These can have very significant practical implications for example for pile construction and temporary slope stability. 5. DBC has a relatively simple microstructure and is characterized by low water content, high density, low void ratio and permeability. 6. The past stress history of the deposit is poorly understood due to the depositional environment which included a high degree of shearing. Normal stiff clay correlations involving classical preconsolidation stress are inappropriate. The in situ horizontal stress coefficient (K_0) is not well defined. 7. High quality samples of DBC can be obtained using triple tube rotary techniques with polymer flush. 8. G_{max} of DBC is very high and can be determined reliably from inexpensive MASW surface wave studies. These tests together with measurements during triaxial tests with sample-mounted transducers can give the full stiffness - strain profile for DBC. 9. Comparison of field (horizontal) permeability values and lab (vertical) values suggest the material may exhibit some degree of anisotropy of permeability.

10. On shearing the material dilates strongly and therefore it is not straightforward to determine s_u .

11. Triaxial CIUC or CAUC tests on these high quality cores give similar s_u values to simple UU tests.

12. DBC possesses a high degree of anisotropy of undrained shear strength with s_u in triaxial extension being about 25% that in compression.

13. In current practice s_u is determined from in situ SPT testing. A large range of values with a high degree of scatter is usually obtained from these tests. In future projects it is recommended these are replaced by good

quality lab tests on rotary cored or block samples.

14. DBC has low sensitivity of effective stress strength parameters with peak and large strain values being relatively similar.

15. It also appears to have a curved effective stress strength profile, which may be indicative of the development of failure along some fissures or joints within the specimen.

10 RECOMMENDATIONS FOR FUTURE WORK

Important future works in Dublin will involve tunneling and deep excavations (e.g. the central Dublin

section of the Metro project). For these works presently poorly understood parameters such as in situ

horizontal stress and anisotropy of stiffness have important implications. Therefore:

1. Work should be carried out in the lab using recently developed techniques such as lateral stress

oedometers to determine K_0 . As has been reported by other researchers the till may exhibit anisotropy

of horizontal stress and measured values of K_0 need to be related to the direction of ice flow.

2. Bender element testing with multiple shear wave directions on high quality cores could give useful data on stiffness anisotropy though work will have to be carried out on techniques for installing the bender in the specimen.

3. Similarly future field investigations should include hydraulic fracture testing and various complementary geophysical techniques.

4. Work should be carried out to determine the implications of anisotropy of s_u on the design and performance of engineering works.

ACKNOWLEDGMENTS

The authors are particularly grateful to George Cosgrave, Senior Laboratory Technician at UCD, who

carried out much of the laboratory testing and to Shane Donohue of UCD for the MASW survey results.

Colleagues at GCG, particularly David Hight and George Milligan have provided useful insights into

the behaviour of DBC and other stiff clays. The authors would also like to thank NMI Joint Venture and

Dublin City Council for permission to publish the DPT data.

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Characterization and engineering properties of

Recife soft clays - Brazil

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ABSTRACT: Comprehensive research has been carried out in Recife soft clay deposits, northeastern

Brazil, since 1978, by the Geotechnical Group of the Federal University of Pernambuco, Brazil. This

paper presents an updated summary and some recent technical information from the studies carried

out including geology, comparison of sampling techniques, physical properties, results of laboratory

mechanical tests, influence of organic material on the properties, results of in situ tests and engineering

properties. The extensive investigation has been complemented by instrumented lateral and vertical pile

loading tests and case histories of a shallow foundation and embankment on soft clay. The materials

investigated are representative of an extensive area and there are also similarities in the characteristics

and engineering behavior with many deposits in Brazil and other countries. Therefore both the local and

global importance of the Recife clay deposits is significant.

1 INTRODUCTION

Recife City is situated on the Northeastern coast of Brazil (Fig. 1) and presents a wide coastal plain and

lowland area. Soft clay and organic soil deposits can be found in about fifty percent of the area.

The Geotechnical Group of the Department of Civil Engineering of the Federal University of Per

nambuco has developed a research program in the Recife soft clays deposits since 1978, performing

laboratory and in situ tests for many sites of the plain area, including two research sites (RRS1 and

RRS2) and the formation of an extensive data base (Coutinho and Oliveira, 1997, Coutinho et al, 1998a,

1999, Coutinho & Oliveira, 2002). The RRS1 (International Club) is located near the center of the city

and the RRS2 (SESI-Ibura) is located near the Recife International Airport. In RRS2 a geotechnical

accident occurred, in 1995, causing total destruction of a one-floor structure on a steel pile foundation.

The location of the investigated sites including the two research fields are also shown in the map of

Recife in Figure 1.

The standard investigation program consists of SPT profile and laboratory characterization tests,

oedometer incremental loading and triaxial UU-C. At the research sites, or when possible in practical

projects, more detailed investigations and researches in engineering problems have been performed,

including complementary laboratory tests, campaign of in situ tests, such as, piezocone - CPTU,

Marchetti dilatometer - DMT, Ménard pressuremeter - PMT and Field Vane, along with studies of

foundations and an embankment on soft soil deposits.

This paper presents an updated summary and some recent technical information from the studies carried

out including geology, comparison of sampling techniques, physical properties, results of laboratory

mechanical tests, influence of organic material on the properties, results of in situ tests and engineering

properties (research and case histories). The materials investigated are representative of an extensive area;

there are also similarities in the characteristics and engineering behavior with many deposits in Brazil and

other countries. Therefore both the local and global importance of the Recife clay deposits is significant.

2 ENGINEERING GEOLOGY

The geomorphology of the Recife area is characterized by two important extensive areas, the surround

ing hills with altitude in the range of 50-100 m and the coastal plain area with altitude in the range of

2-9 m (Fig. 1).

Figure 1. Location of Recife - Pernambuco and the investigated sites including the two research sites.

Figure 2. Geologic map of Recife City (Alheiros 1998).

Figure 3. Typical geotechnical profiles - Recife lowland area (Coutinho & Oliveira 2002, Bello 2004).

The sedimentary coastal plain area formed in the Quaternary period has a rich history of deposition,

showing wide types of deposits, where the soil profile and geotechnical characteristics are very closely

linked to the agents that control the erosion and deposition, in this case fresh and salt water (river and sea),

wind, gravity, organic material and organisms. A significant variation in the soil profile can be observed

in small distance. In general, the tendency is for the thickness of the soft soil deposits to increase from

the coast to the interior. Figure 3 presents typical geotechnical soft soil profiles from the Recife city lowland area. The presence

of a fill/sand layer in the upper soil, the stratification of the soft soils deposits, and the existence of sandy

clays and/or organic soils can be observed. In general, the clays consistency is soft (SPT <4), but layers

with medium consistency also occur. The water table is usually located between 0 and 2 m depth.

The soft clays deposits are geologically characterized to be situated between two marine terraces

formed during the penultimate transgression (Pleistocene

Period) and the last transgression (Holocene

Period) of the sea (Fig. 2). These deposits were deposited 10,000 (maximum) years ago in an alluvial

lacustrine environment, with marine and mangrove influence, localized in lower levels, probably

receiving fine sediments of zones of the Barreiras and Cabo Formation (Alheiros 1998). The land

level is close to sea-level, thus the soft soil deposits, in general, are almost totally below the water table.

The importance of human action in the actual conformation of the deposit can be observed in successive

embankments performed with time, preparing the land for utilization in supporting the foundations of

buildings and other engineering workmanship, at least since the period of the Netherlands colonization

(1630-1654). It is estimated that a total area of 25 million square meters of embankment was constructed.

3 GENERAL GEOTECHNICAL PROFILE

The two Recife research sites, which consist of a 20 m thick deposit of high plasticity soft clay, have

been described in numerous research projects (CNPq) and associated theses under the coordination of

the author. The geotechnical properties of the two deposits are detailed in Figures 4 and 5. The RRS1

site presents practically two different homogeneous soft layers ($SPT < 4$) with thickness of 10 m each.

Both layers show a clay fraction around 65%. The average plasticity index, I_p , is 70% in the first layer

and 33% in the second layer. The natural water content in the first layer varies with depth in the range of

65-100%. In the second layer the natural water content is in the range of 45-65%, initially decreasing

slightly with depth and afterwards increasing with depth.

The average liquidity Index, IL, is 0.66 (first

layer) and 0.71 (second layer). The undrained shear strength measured with the field vane basically

shows values in the range of 35-55 kPa, with three different intervals, increasing with depth in each one.

The average sensitivity, St, determined with the field vane is about 6.5 (layer 1) and 13.0 (layer 2). D e p t h (m)
 Description SPT Grain Size (%) Water content and Limits (%)
 I L Shear strength (kPa) Effective stress (kPa) 5 _ 10 _ 15
 _ 20 _ 25 _ Clay/Sand 4/45 0 . 6 6 ± 0 . 0 8 0 . 7 1 ± 0 .
 1 0 Sandy/ Clay O r g a n i c S o f t C l a y (1) O r g a
 n i c S o f t C l a y (2) 2/45 2/54 2/55 2/54 1/46 2/58
 4/54 2/52 2/48 3/61 3/46 20 0 50 100 WL WP W 10 20 0 50 100
 Fine sand Silt Clay 10 20 0 10 20 30 40 50 60 70 Su vane Su
 disturbed w w w 20 0 100 200 s'vo s'p σ vo σ' p

Figure 4. Geotechnical properties of RRS1 clay deposit. The preconsolidation pressure profile deduced from conventional 24 hour oedometer tests presents

an average value of about 150 kPa. In the first layer, the preconsolidation pressure initially decreases

slightly and then increases with depth, indicating an overconsolidation ratio, OCR, decreasing from

around 2.5 at a depth of 6 m to a constant value around 1.4 in the depth of 11-16 m. In the second

layer, the preconsolidation pressure slightly increases with depth indicating a constant OCR of about

1.1. The water table fluctuates seasonally between 1-2 m depth. At depth, ground water conditions are

hydrostatic. The RRS2 site presents initially a small layer of an organic soft soil and afterwards a thick soft clay

deposit with two different layers (SPT <W/220 - hammer weight/220 cm of penetration) with thickness

of 7.5 m and 10 m, respectively. Both layers show a clay fraction around 72%. The average plasticity

index, Ip, is 98% in the first layer and 53% in the second layer. The natural water content in the first

layer averages 150%. In the second layer the natural water content averages 84%. The average liquidity

Index, IL, is 0.90 (first layer) and 0.81 (second layer). The undrained shear strength measured with the

field vane shows values in the range of 15-37 kPa, with two different layers, initially decreasing with

depth and afterward remaining constant in the first layer, and in the second layer increasing with depth.

The average sensitivity, S_t , determined with the field vane is about 6.1 (layer 1) and 10.9 (layer 2).

The preconsolidation pressure profile deduced from conventional 24 hour oedometer tests presents an

average value of about 40 kPa. In the first layer, the preconsolidation pressure initially decreases slightly

and then increases with depth, indicating an overconsolidation ratio, OCR, decreasing from around 2.5

at a depth of 4 m to a constant value around 1.0 in the depth of 8-11.5 m. In the second layer, the

preconsolidation pressure slightly increases with depth indicating a practically normally consolidation

condition ($OCR \approx 1$). The water table is located in the range of 0 to 1 meter. Artesian pressure and

gas pressure also are observed inside of the very soft clay layers showing higher pore-pressure than the

hydrostatic conditions, slightly reducing the overburden effective stress.

4 COMPOSITION

Figures 4 and 5 show geotechnical and composition profiles of the research sites. In RRS1 the soft clay

composition is an average of 65% of clay, 25% of silt and 10% of sand, showing a smaller value at a

depth of 20 m. In RRS2 the soft clay composition is an average of 72% of clay, 20% of silt and 8%

of sand. In both Recife research sites (RRS1 and RRS2) the main clay mineral is kaolinite with the possible

presence of a more active clay mineral. In RRS1, the molecular relationship K_i (silica/alumina) is 2.5 to 3.1, SiO_2 and Al_2O_3 in the range

of 23 to 35 % and 14 to 20%, respectively. The values of $K_i = (SiO_2)/(Al_2O_3)$ and $K_r = (SiO_2)/(R_2O_3)$ Depth (m) Description SPT Grain Size (%) Water content and Limits (%) I L Shear strength (kPa) Effective stress (kPa) 5 5 4 Org. Soil 3/40 5 _

10 _

15 _

20 _ 3/33 7 Clayey/ Sand Organic Silty Clay (2) W/100 W/230 W/200 W/215 W/200 W/195 Fill Organic Silty Clay (1) 0.90 ± 0.16 0.81 ± 0.17 0.5 10 15 20 0.5 100 150 σ'_{vo} σ'_p 0.5 10 15 20 0 10 20 30 40 50 Su vane Su disturbed 0.5 10 15 20 0 50 100 Sand Silt Clay 0.5 10 15 20 0 100 200 WL WP W w L w P w

Figure 5. Geotechnical properties of RRS2 clay deposit.
*Note: number of blows/penetration of the sampler.

Figure 6. Results of organic content. (a) RRS1 (Coutinho & Oliveira 1997), (b) RRS2 (Coutinho et al. 1999).

indicate a low degree of chemical weathering. The soluble salt content is high indicating a fluvial-marine

deposit. The clay shows low permeability in the order of 10^{-9} to 10^{-10} m/s.

The presence of marine shells is observed in the clay deposits in both research sites (marine influence).

The determination of the organic matter content (OC) was done by two methods (see Coutinho 1986):

(a) chemical method consisting in oxidizing the organic matter with a mixture of potassium dichromate

and sulfuric acid; (b) ignition loss by heating a sample previously dried at $105^\circ C$ at $400 \pm 5^\circ C$ during

12 hours. Figure 6 shows the OC values obtained for both research sites. In RRS1 the values are in the range of

3% to 9%, with layer 1 generally showing higher values. In the RRS2 site the organic content values for

the soft clays are in general situated in the range of 3-11% (depending on the method), showing slightly

Figure 7. Plasticity chart - Recife and Juturnaíba soft soil results (from Coutinho et al. 1998b).

higher values (in the order of 13-14 %) at the depth of 8.5 m. In the organic soil layer (3-4 m depth) the

values are in the range of 40-60%.

5 STATE AND INDEX PROPERTIES

5.1 Atterberg's Limits

Results of the Atterberg Limits and soil profile from the two research sites are presented in Figures 4

and 5. The Atterberg Limits (w_L and w_P) were measured without oven drying, since Brazilian experience

shows that I_p tends to drop by 10 to 30% with oven drying. In RRS1 the w_L and w_P of the first soft layer (6-16 m) present values in the range of $98 \pm 28\%$ and

$40 \pm 11\%$, respectively. In the second soft layer (16-26 m) the w_L and w_P values are in the range of

$65 \pm 10\%$ and $30 \pm 5\%$, respectively. The plasticity index of the first soft layer (6-16 m) presents values

in the range of $70 \pm 2\%$, while in the second soft layer (16-26 m) the values are in the range of $33 \pm 6\%$. In RRS2 the w_L and w_P of the first soft layer (4-11.5 m) present values in the range of $165 \pm 25\%$

and $67 \pm 17\%$, respectively. In the second soft layer (11.5-21 m) the w_L and w_P values are in the range

of $93 \pm 8\%$ and $40 \pm 6.07\%$, respectively. The plasticity index of the first soft layer (4-11.5 m) presents

values in the range of $98 \pm 14\%$, while in the second soft layer (11.5-21 m) the values are in the range

of $53 \pm 6\%$. As expected, the peat layer presents higher

values to the plasticity index (111%-one result). Figure 7 presents a plasticity chart with results from laboratory tests of Recife soft / medium clays

and organic soils / peats deposits. Results from another Brazilian organic clay / soil deposit (Juturnaíba

Dam in the state of Rio de Janeiro) are also presented. The Recife soft soils were divided into four

groups (Perrin 1974, Magnan 1980): sandy and silty clays ($OC \leq 3\%$), organic clays ($3 < OC \leq 10\%$)

and organic soils ($10 < OC < 30\%$)/peats ($OC \geq 30\%$), by using the creation of subgroups tools. In the

chart, proposed ranges for inorganic and organic clays and peats are also included, as presented by

Coutinho & Lacerda (1987). It can be observed in this chart that the results of the Recife soft / medium clays are around the A-line,

with liquid limit (w_L) values ranging from 23% to 235% and plasticity index (I_p) values ranging from

5% to 148%. The results of the Recife and Juturnaíba organic soils are below the A-line around the

proposed ranges. The w_L is between 175% and 235% and (I_p) between 40-120% (Recife). Some results

of Atterberg Limits are not shown in this figure because of difficulties to performing the tests in some

of the organic soils - peats.

5.2 Void ratio

Figure 8 presents the soil and initial void ratio profiles from the two research sites. In RRS1 the initial

void ratio of the first soft layer (6-16 m) presents values in the range of $2.06 \pm 0.36\%$, while in the

Figure 8. Initial void ratio (a) RRS1 (Coutinho & Oliveira 1997), (b) RRS2 (Coutinho et al. 1999).

second soft layer (16-26 m) the values are in the range of $1.35 \pm 0.19\%$. In RRS2 the initial void ratio

of the first soft layer (4-11.5 m) presents values in the range of $3.97 \pm 0.61\%$, while in the second soft

layer (11.5-21 m) the values are in the range of $2.05 \pm 0.22\%$. Statistical correlation of e_0 vs. $w(\%)$ has been developed using all data from the laboratory tests

database, to help research and practical projects. The results are shown on Figure 11. Results of the

Juturnaíba organic soils (Coutinho 1986, Coutinho & Lacerda 1987) are also shown and compared

with the Recife results. The initial void ratio (e_0) values of the Recife soils are between 0.5 and 5.25

(soft/medium clays) and between 3.45 and 14.4 (organic soils/peats).

The e_0 vs. $w(\%)$ correlation is, in general, similar when Recife and Juturnaíba deposits are compared

(especially for clay deposits), with a higher dispersion for Recife organic soils. Due to the quality of some

samples for the Recife organic soils, the values show more dispersion but maybe the relation presents a

higher inclination. In general, the clay correlations present higher correlation coefficients (r^2) and lower standard error

(lower dispersion) than the organic soil/peat correlations. The organic soft soils of Juturnaíba research

site present higher correlation coefficients than the Recife deposits, as would be expected (better quality

of samples).

5.3 Water content

Figures 4 and 5 presents the soil and water content profiles from both research sites RRS1 and RSS2. In RRS1 the natural water content is usually slightly below the liquid limit in both layers, showing

values in the range of 65-100% in layer 1 and in the range of 45-65% in layer 2. In RRS2 the natural water

Figure 11. Liquidity index (a) RRS1, (b) RRS2.

soft layer (16-26 m) the values are in the range of $1.70 \pm 0.05 \text{ Mg/m}^3$. The density of solids (ρ_s) on the

first layer presents values in the range of $2.65 \pm 0.02 \text{ Mg/m}^3$, while in the second soft layer the values

are in the range of $2.64 \pm 0.10 \text{ Mg/m}^3$. In RRS2 the density (ρ) of the first soft layer (4-11.5 m) presents values in the range of $1.32 \pm$

0.06 Mg/m^3 , while in the second soft layer (11.5-21 m) the values are in the range of $1.47 \pm 0.04 \text{ Mg/m}^3$.

The density of solids (ρ_s) on the first layer presents values in the range of $2.50 \pm 0.09 \text{ Mg/m}^3$, while in

the second soft layer the values are in the range of $2.61 \pm 0.05 \text{ Mg/m}^3$.

5.6 Liquidity index

Figure 11 presents results of liquidity index IL obtained in the research sites. In RRS1 the values of

the liquidity index of first layer (6-16 m) and of the second soft layer (16-26 m) are in the range of

0.66 ± 0.08 and 0.71 ± 0.10 , respectively. In RRS2 the liquidity index of the first soft layer (4-11.5 m)

presents values in the range of 0.90 ± 0.16 , while in the second soft layer (11.5-21 m) the values are in

the range of 0.81 ± 0.17 .

6 EVALUATION OF SAMPLE QUALITY

Efforts in the research projects have been made to understand, quantify, minimize and, if possible, correct

the effect of disturbance in the Recife softy clays geotechnical parameters (Coutinho 1976, Ferreira 1982,

Ferreira & Coutinho 1988, Coutinho et al. 1998c). Usually, in the research works, sampling has been done by means of a stainless steel thin-wall tube

(Shelby) or stationary piston sampler, with 800-1000 mm long, internal diameter of 100-110 mm, area

ratio of 7%, respectively. Steps are taken to minimize the disturbance of the samples in the field and in

the laboratory and very good quality sample have been obtained. Sherbrooke block sampling (Lefevre & Poulin 1979) was carried out in March 1999 in the RRS1 (joint

research between Civil Engineering Program of Federal University Rio de Janeiro and Federal University

Pernambuco - Norwegian Geotechnical Institute), with the purpose of continuing this investigation using

very high quality samples in laboratory studies (Oliveira et al. 2000, Oliveira, 2002).

The behavior of Recife clays (RRS1) under one-dimensional consolidation is heavily affected by

sampling disturbance. Figure 12 presents examples of comparative oedometer curves of samples taken

Figure 12. Oedometer curves from samples taken in different ways - RRS1 site (Coutinho et al. 1998a, Oliveira

2002).

Figure 13. (a) Influence of the sample quality: (a) c_v vs $\log p$; (b) S_u profile - laboratory and vane test - RRS1

(Coutinho et al. 1998c).

by different samplers and quality: Sherbrooke, Shelby 100 and 60 mm. Also shown (Fig. 12b) is the

correspondent oedometer curve for a complete disturbed sample obtained in the laboratory. In general,

the Sherbrooke and Shelby 100 mm presented very good results (Fig. 12a). In some cases, the block

samples showed the best results, as can be seen in Figure 12b. The relation between preconsolidation pressures between good and bad quality samples was found to

be as high as 3, although initial void ratios do not seem to be significantly influenced by sampling quality

(Coutinho et al. 1998). It is also difficult to obtain preconsolidation pressure values from compression

curves from bad quality samples. Significant reductions in values of the compression index C_c and the Table 1. Sampling effect on one-dimensional compression for Recife and Sarapuı́-RJ clays (Coutinho et al. 1998c). Sarapuı́ - Rio Recife (RRS1) de Janeiro G/B G/L G/B G/L σ'_{pconv} 1.5-3.0 3.0-5.0 1.5-2.0 1.5-2.5 C_s 0.8-1.0 0.7-1.2 0.9-1.2 1.0-1.1 C_c 1.2-2.0 1.2-2.1 1.2-1.5 1.4-1.7 c_v (NC range) 1.21 1.93 1.26 1.37 G = good quality samples; B = bad quality samples; L = laboratory remoulded samples

Figure 14. Sample quality classification for the studied sites (from Coutinho et al. 1998c and Oliveira 2002).

coefficient of vertical consolidation c_v were also observed as the soil was destructured by bad quality

sampling (see Fig. 13a and Table 1). The swell index C_s in general showed a slight increase with sampling

disturbance. Comparison of undrained strength results from laboratory tests (Triaxial unconsolidated undrained

compression (UUC) and isotropically consolidated undrained compression (CIUC) tests) and vane field

tests obtained by Teixeira (1972), Oliveira (1991) and Oliveira (2000) for the RRS1 site is also shown in

Figure 15b. The smaller S_u values obtained by Teixeira (1972) are due to the disturbance of the samples

(sampling by Shelby with diameter of 60 mm) and conditions of the tests (procedure and equipment) and

show reductions in the order of 50% in comparison for those obtained by Oliveira (1991) and Oliveira

(2000). Many researchers have investigated means of understanding and combating these disturbance pro

cesses. A recent discussion about tube sampling effects based on the results of constant rate of strain

Table 2(a). Criterion of sample quality classification (Lunne et al. 1997), (b) Sample quality classification - Recife

soft clays (Coutinho et al. 1998c, Oliveira 2002) (a) (b)

$\Delta e/e_0$ $\Delta e/e_0$

C C

R Very good Good to Very R Very good Good to Very to
excellent Fair Poor poor to excellent Fair Poor poor

1-2 <0.04 0.04-0.07 0.07-0.14 >0.14 1-2.5 <0.05 0.05-0.08
0.08-0.14 > 0.14

2-4 <0.03 0.03-0.05 0.05-0.10 >0.10

Figure 15. (a) Compression ratio CR (%) vs. $e_{vo} - RRS2$;
(b) OCR vs. $e_{vo} - RRS2$ (from Cavalcante et al. 1998a).

consolidation tests and unconsolidated undrained triaxial
compression tests is presented in Santagata

et al (2006). A quantitative procedure on the evaluation of
the quality of samples has been defined by NGI -

Norwegian Geotechnical Institute (Lacasse 1988). This
procedure uses the volumetric deformation (ϵ_{V0})

corresponding to the initial effective vertical stress (σ'_{V0}). This criterion was later modified using the

ratio $\Delta e/e_0$ by Lunne et al. (1997) and it is shown in
Table 2a. For a particular clay deposit multiply

$\Delta e/e_0$ by $e_0/(1 + e_0)$ to get the criteria in terms of ϵ_{V0} . Figure 14a shows the volumetric strain (ϵ_{vo}) for σ'_{V0}
profile obtained in oedometer tests from both

research sites, including also results from Sherbrooke and
completely remolded samples. The straight

line presented corresponds to the limits suggested by Lunne
et al. (1997), separating the specimens,

on the left side, presenting at least a fair quality of
sampling. Considering the local experience, this

proposal seems to be too rigorous for the Recife organic
soft clays (see also Santagata et al 2006).

Table 2b presents criterion for sample quality

classification proposed for the Recife soft soils. Coutinho

et al. (1998c) also show results of the volumetric strain (ϵ_{vo}) for σ'_{vo} for deposits from Rio de Janeiro

(Juturnaíba and Sarapuí), where it was possible to obtain very good quality samples. Figure 15 shows the compression ratio (CR) and OCR versus ϵ_{vo} for RRS2. The chart indicates, as

expected, that the CR and OCR values strongly decrease when ϵ_{vo} increases. There is a lower limit to

the CR value (20%) and to the OCR value (0.25), when the sample is almost totally disturbed. This type

of correlation can be useful for an approximate correction of the CR and OCR values, considering the

quality of sampling, in practical projects. ϵ_{vo} values for criterion for evaluation of sample disturbance are

also shown.

7 ENGINEERING PROPERTIES

7.1 In situ tests

The following in situ tests were carried out at the two research sites: SPT, piezocone - CPTU, Marchetti

dilatometer - DMT, Ménard Pressuremeter - PMT, and Field Vane test. (a) (b) 0 5 10 15 20 D e p t h (m) 5 Fill 8 1 7 1/33 Organic Soil/ Peat Organic silty clay Organic silty clay P/141 P/108 1/31 P/24 1/34 P/134 P/131 1/39 6 6 Medium silty sand 1 1/34 1 1 1 WT Soil Profile 0 20 40 60 80 100 120 140 160 180 200 220 240 260 280 300 340 320 W(%) SPT02 W Water Content (%) 0 5 10 15 20 0 100 200 300 Natural Water Content W(%)

D

e p

t h

(m

) Shelby Sampler SPT Sampler peaty soil 2nd soft clay layer
1st soft clay layer

Figure 16. Results of natural water content from laboratory and in situ (SPT) samples. (a) RRS2, (b) Practical

work - Recife. In many countries, including Brazil, the SPT is still the major in situ test for basic site characterization.

The author suggests that attention should be drawn to how important it is to include measurements of

natural water content, with a standard procedure during the SPT test, to improve the ground charac

terization in soft clays/organic soils deposits. Figure 16 illustrates this proposal showing water content

results from SPT samples from RRS2, including reference laboratory water content results from Shelby

tube samples.

The equipment and procedures for the piezocone and the Marchetti dilatometer testing performed in

the studies were in the accordance with what is suggested in the literature (e.g. ASTM 1986, Schmertmann

1988, Campanella & Robertson 1988). Both equipment were pushed into the ground at an approximate

rate of 20 mm/s, using a CPT rig, with a capacity of 5 to 6 tons. The dilatometer was stopped every 200 mm

and pressure measurements were taken. The piezocone prototypes used were the COPPE Mark-III, with

one measurement of pore pressure (Oliveira 1991, Coutinho & Oliveira 1997), and the COPPE Mark-IV,

with two measurements of pore pressure on the face (u_1) and behind the cone (u_2) (see Bezerra 1996,

Coutinho et al. 1999). Dissipation tests were performed at some depths using both equipment (CPTU

and DMT). The piezocone and the dilatometer tests, performed in the Recife soft clays, represent a joint

effort of the Geotechnical Groups of the UFPE, COPPE/UFRJ

and Geomecânica Company.

The Ménard pressuremeter tests (PMT) were carried out through a joint research between the Geotech

nical Groups of the UFPE and Federal University of Paraíba (Cavalcante 1997, Cavalcante et al. 1998b).

The procedure used was in accordance with French Standard (NF-P94/110 1991).

The field vane tests were performed using the new electrical equipment developed and manufac

tured through a joint project between COPPE/UFRJ, GROM Company, and UFPE. This equipment has

the torque cell located near the vane to minimize/eliminate internal and external friction. The proce

dure used was in accordance with the Brazilian Standard (NBR 10905 1989) that follows international

recommendations.

All in situ tests were well performed and the results were good. Typical results and index parameters

from the piezocone tests are shown in Figure 17 (RRS1) and in Figure 18 (RRS2). Due to unbalanced pore

water pressure, a correction was applied to the original recordings of cone resistance and sleeve friction,

using the proposal of Lunne et al. (1985) and Konrad (1987). Three tests were performed and presented

similar results with very good repeatability, identification of soil type, stratigraphy, and classification.

The results of dilatometer index parameters for the three tests performed, at each site, are shown

in Figures 19 and 20. They also presented very good repeatability and identification of soil type and

classification.

Figure 17. Typical piezocone results - RRS1 (Oliveira 1991,

Coutinho et al. 1993).

Figure 18. Typical piezocone results - RRS2 (Coutinho et al. 1998a).

Figure 19. Dilatometer tests results - I_D , K_D , E_D , U_D vs Depth - RRS1 (Coutinho & Oliveira 1997).

Figure 20. Dilatometer tests results - I_D , K_D , E_D , $U_D = (p_2 - u_0)/(p_0 - u_0)$ vs Depth - RRS2 (Coutinho et al. 1999).

Figure 21. Classification by Robertson (1990) - RRS1 and RRS2 deposits (Coutinho et al. 1993, 1998). Results of the field vane tests are shown in the Figures 4 and 5. Results of the pressuremeter tests are

presented later in this paper. The location of the RRS1 and RRS2 results in the soil behavior classification chart proposed by

Robertson (1990) is detailed in Figure 21 which is in accordance with the chart. Figure 22 summarizes the location of the soils tested by NGI on the dilatometer soils classification

chart proposed by Marchetti & Crapps (1981) and modified by Lacasse & Lunne (1988). The newer

information enables one to illustrate qualitatively the effects of overburden, overconsolidation ratio and

density on the dilatometer modulus. For Norwegian soils, material indices between 0.05 and 0.1 have

been obtained. The original chart was therefore extended in this direction. The position of the Recife

soft clays deposits are also presented in Figure 22 and they are in accordance with the chart.

7.2 Yield stress (or preconsolidation stress)

Laboratory incremental oedometer tests were performed in soil specimens with a diameter of 87 mm, a

height of 20 mm, using a Bishop's type apparatus, with double drainage. Loads were doubled for each

stage, beginning with 5 to 10 kPa until 640 to 1280 kPa and

then decreased down to 10 kPa. Each stage

usually lasted 24 hours. Figures 23 and 24 show the results of the effective overburden stress (σ'_{V0}) and the preconsolidation

pressure (σ'_p) determined by Casagrande's method, including Sherbrooke samples. In the first layer of the RRS1 site, the preconsolidation pressure initially decreases slightly and

then increases with depth, showing on overconsolidation condition ($\sigma'_p > \sigma'_{V0}$). In the second layer, σ'_p

increases with depth with average values slightly higher than σ'_{V0} . In the RRS2 site, the preconsolidation

pressure initially decreases with depth, showing on overconsolidation condition and then from 8.0 m it

increases with depth with values in the same order as σ'_{V0} , practically a normally consolidation condition. In some specimens from all soft layers, the preconsolidation pressure (σ'_p) was also determined accord

ing to the deformation at the end of primary consolidation ("d 100"). The results present higher values

Figure 22. Classification chart for soil test. Effects of overburden, overconsolidation ratio and density (Lacasse &

Lunne 1988) with results of Recife soft clay deposits - RRS1 and RRS2 (Coutinho et al. 2006).

Figure 23. Stress history and in situ horizontal parameters - RRS1 (from Coutinho & Oliveira 1997) including field

vane results (Oliveira 2000).

Figure 24. Stress history and in situ horizontal stress parameters - RRS2 (from Coutinho et al. 1999), including

vane results (Oliveira 2000).

than the conventional procedures ("d f" - the end of stage, Tables 3 and 4): RRS1 - σ'_p ("d 100") $> \sigma'_p$ ("d f"):

3-26% (average 16%); RRS2 - σ'_p ("d 100") $> \sigma'_p$ ("d f"): 26% in average. The discussion about the potential of

in situ tests to obtain σ' / P can be seen below in the section on

overconsolidation ratio (OCR).

7.3 OCR

Figures 23 and 24 show results of the overconsolidation ratio (OCR) obtained from laboratory oedometer

tests, for both research sites. Figure 23 (b and c) shows for RRS1, a small overconsolidated upper crust

with OCR values decreasing from a value of about 3.0 to 1.3, and remaining approximately equal to 1.3

until reaching layer 2 which is essentially normally consolidated ($OCR \approx 1.15$). Figure 24 (b and c) shows a similar pattern for RRS2 with layer 1 having an overconsolidated crust.

However, the OCR values decrease more rapidly from about 3.0 to values close to 1.0 at 8.0 m depth.

Layer 2 at RRS2 is practically normally consolidated ($OCR \approx 1.0$). The behavior described for the two research sites is in general characteristic for all Recife soft clays

deposits (plain area). The overconsolidation condition in the upper part of the clay deposits is considered

to be due to desiccation, aging and water table fluctuation. Below, it is essentially normally consolidated. OCR values can be heavily affected by sampling disturbance (see Table 1 and Fig. 15). OCR values were obtained using correlations proposed in the literature from in situ tests (vane,

dilatometer and piezocone tests):

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. Laboratory Piezocone Average Piezocone (1) Piezocone (2) OCR OCR* (a) Average DMT (3) σ_v OCR Lunne et al. (1989) Kulhawy & Mayne (1990) OCR OCR OCR* Vane (4) Depth. Sully et al. (1988); OCR* Layer (m) (d100) $df/d100$ Q_{tu}/σ'_v $B_{qt}(qT - \sigma_v)$ uface (1) (2) Sully & Campan. (1992) (b) (c) (d) 1 8.02.71.88/2.182.111.003.002.1 22.191.252.031.852.452.022.221 .559.23.71.50/1.742.121.381.93 2.132.202.001.812.112.892.162. 381.6511.43.01.44/1.671.731.20 1.701.731.781.811.541.772.091. 842.031.3913.45.61.38/1.601.52 1.001.731.511.561.541.421.542. 091.491.641.40219.24.21.06/1.2 31.261.001.531.311.351.481.261 .381.400.951.051.2520.75.21.06 /1.231.401.001.541.331.371.461 .311.391.400.971.071.2622.66.0 1.06/1.231.521.001.741.561.601 .561.421.571.400.971.071.4524. 36.01.06/1.231.541.001.691.501 .521.571.411.531.400.951.05 - Ta ble 3 (b). Results of OCR - Laborator y, piezocone, dilatometer and fiel d vane tests: RRS2 (Coutinho & Olive ira 2002; Coutinho et al. 2006). Lab oratory Piezocone Average Piezocone

$ne(1) Piezocone(2) OCR OCR * (a) Average DMT(3) \sigma'_{vo} OCR Lunne et al. (1989) Kulhawy \& Mayne (1990) OCR OCR OCR * Vane(4) Depth. Sully et al. (1988) ; OCR * Layer(m) (d100) df / d100 Q_{tu} / \sigma'_{vo} B_{qt} (q_T - \sigma'_{vo}) u_{face}(1)(2) Sully \& Campan. (1992) (b)(c)(d) 18.02 .71.88 / 2.182.111.003.002.122.191.252.031.852.452.022.221.559 .23.71.50 / 1.742.121.381.932.132.202.001.812.112.892.162.381.6511.43.01.44 / 1.671.731.201.701.731.781.811.541.772.091.842.031.3913.45.61.38 / 1.601.521.001.731.511.561.541.421.542.091.491.641.40219.24.21.06 / 1.231.261.001.531.311.351.481.261.381.400.951.051.2520.75.21.06 / 1.231.401.001.541.331.371.461.311.391.400.971.071.2622.66.01.06 / 1.231.521.001.741.561.601.561.421.571.400.971.071.4524.36.01.06 / 1.231.541.001.691.501.521.571.411.531.400.951.05 - (*) Note: (a) Piezocone(2): Kulhawy \& Mayne (1990) average of results: $OCR = 0.32 \times Q_t$; $\sigma' P = 0.33 \times (q_T - \sigma'_{vo})$; $\sigma' P = 0.47 \times u_{face}$; (b) DMT(3): Lunne et al. (1989) average of results: $OCR = 0.30 \times K_1, 17D$; (c) DMT(3): Lunne et al. (1989) average of results: $OCR = 0.33 \times K_1, 17D$; (d) Field vane(4): Mayne \& Mitchell (1988). Equation 3 has built-in the assumption that $K_D = 2$ for $OCR = 1$. This assumption has been confirmed$

in many genuinely NC (no cementation, aging, structure) clay deposits (Marchetti et al. 2001).

OCR values were also obtained from the piezocone test results through the empirical correlations proposal

by Lunne et al. (1989). Figures 23 and 24 and Tables 3 and 4 also show results of the OCR obtained from correlations using

in situ tests. The OCR values obtained from field vane tests by using the Mayne \& Mitchell (1988)

Equation 1 are close to the laboratory results, except for

the dried crust in RSS1. Soares et al. (1997) got

generally good agreement among the data derived from oedometer and vane test, for the Porto Alegre

clays. Tables 3a and 3b show the results obtained for OCR at eight depths in the soft deposit, for the two

studied sites. The volumetric strain (ϵ_{vo}) for σ'_{vo} is also presented giving an idea of the quality of the

soil specimens tested. In general, the results of OCR from in situ tests correlations are similar and in

the same order as the range of the laboratory results. Lunne et al. 1989 and Kulhawy & Mayne 1990

correlations from piezocone results presented OCR values slightly higher than values from laboratory

tests, particularly if OCR from d 24hour is considered. The exception is the Lunne et al. (1989) correlation

using u/σ'_{vo} . The difference between each piezocone estimative from the Lunne et al.'s correlation is a

good index of the reliability of the estimated OCR (Sugawara 1988). Demers & Leroueil (2002) presents an important review and discussion about the evaluation of precon

solidation pressure and the overconsolidation ratio from piezocone tests using results from clay deposits

in Quebec. For the sites investigated, the most reliable relationship is the one directly relating precon

solidation pressure to net tip resistance with the parameter $N_{\sigma t}$ [$\sigma'_{p} = (q_t - \sigma_{VO}) / 3.4$] and accordingly,

the one relating OCR to the parameter Q_t ($OCR = Q_t / 3.4$). Ninety-five percent of the results obtained

are within $\pm 20\%$ of the laboratory measurement of σ'_{p} . Good results were also obtained with excess

pore pressure, especially those measured at the tip apex (position u_1) - $\sigma'_{p} = 0.37 (u_1 - u_0)$. Finally, the

difference between tip resistance and the pore pressure measured at u_2 gives a reasonable correlation. Considering the results above (OCR from piezocone tests), a preliminary $N_{\sigma t}$ value in the order of

4.0 ($OCR = Q_t / 4.0$) can be an adequate result for Recife clay deposits. If σ'_p for d_{100} is considered

(deformation of primary consolidation) the $N_{\sigma t}$ value (3.4) obtained by Demers & Leroueil (2002) gives

adequate results. Figure 24 and Tables 3a and 3b also present results of OCR calculated from the Sully et al. (1988) and

Sully & Campanella (1992) CPTU correlations and results of oedometer tests for RRS1 and RRS2. The

values obtained from piezocone tests are also slightly higher than those deduced from laboratory tests.

The results obtained from this correlation in general are of the same order as the average values from

CPTU using other correlations. Contrary to the results of Sully et al. (1988), in the Demers & Leroueil

(2002) studies no trend was found between OCR and the PPD 1-2 parameter. The overconsolidation ratio OCR has been usually defined as the ratio of the "maximum" past effective

stress and the currently vertical applied stress. Marchetti (1980) pointed out the similarity between the

K_D and OCR profiles and this was later confirmed by several authors (e.g. Jamiolkowski et al. 1988). In

Table 4. Natural content, I_p , S_{UVANE} and S_t values for Brazilian clays. S_{UVANE} (kPa) S_t S_t

Site W_N (%) I_p (%) (range) (range) (average) Reference

Recife 40-340 18-95 14-20 8.0 (Layer 1) 2.0-15.0 Bello (2004)

(BR-101) 12.0 (Layer 2)

Recife-PE 45-100 33-70 34-56 6.4 (Layer 1) 4.5-11.8 Oliveira (2000)

(RRS1) 13.0 (Layer 2) 9.2-15.8

Recife-PE 80-150 53-96 14-37 6.1 (Layer 1) 4.7-8.2 Oliveira & Coutinho (2000)

(RRS2) 10.9 (Layer 2) 7.8-14.4

the present research this similarity is also very well observed with the “exception” of the upper part of the

first soft clay layer in RRS1. From the K_D profile (Figs 20 and 21) in both research sites the NC layer 2 has

$K_D \approx 2.0$ to 3.0 indicating some level of cementation/structure/aging according to Marchetti et al. (2001).

The values of K_D are lower at RRS2 than RRS1 indicating that the level of cementation/structure/aging

at RRS2 is likely to be less than at RRS1. Figure 25 shows that the correlations for OCR proposed by Lunne et al. (1989), using $m = 0.30 - 0.33$,

and Powell et al. (1988) can be used with reasonable confidence in Recife soft clays. The Marchetti

(1980) OCR correlation presents significantly higher values than the reference values considered in this

research. Lunne et al. (1989) estimated that, for “young” clays, the uncertainty associated with OCR

from DMT is about 30%.

Table 8 shows results of a quantitative comparative study between the values of OCR predicted from

in situ tests and results from the reference tests (laboratory oedometer tests) for Recife soft clays.

7.4 In situ horizontal stress (K_0)

In situ horizontal stress (σ'_h) (or coefficient of earth pressure at rest, K_0) is an important geotechnical

parameter, however, it is very difficult to get good measurements with any device. In general, there is

uncertain reliability, because of the scarcity of referent values (Lunne et al. 1990).

Values of K_0 were obtained from results of piezocone, dilatometer and laboratory tests using

correlations proposal in the literature:

Figures 23 and 24 present for both research sites, the average values of K_0 that were obtained from all

correlations, showing that the DMT results were very close to the “laboratory” and that the K_0 values

from the piezocone are very dependent on the correlation used.

The CPTU correlation proposed by Sully & Campanella (1991) presents very good agreement with

the laboratory results for RRS2 (Fig. 24). Bezerra (1996) (see also Bezerra et al. 1998) found similar

results for RRS1. 0.4 0.5 0.6 0.7 0.8 0.9 1.0 2.0 3.0 4.0 5.0 6.0 K_0 RRS1 RRS2 (1) Lunne et al. (1990) (2) Marchetti (1980) (1) (2) 0 1 2 3 4 2.0 2.5 3.0 3.5 4.0 4.5 5.0 5.5 6.0 K_0 D O C R RRS2 RRS1 (1) Lunne et al. (1989) (2) Marchetti (1980) (3) Kamei and Iwasaki (4) Powell et al. (1988) (3) (2) (4) (1) (a) (b) (1995)

Figure 25. (a) Stress history and in situ horizontal stress parameters; (b) Coefficient of earth pressure at rest K_0

stress parameters (Coutinho et al. 2006).

Figure 26. Typical e vs $\log \sigma'_v$ curves - oedometer tests. (a) RRS1; (b) RRS2. Lunne et al. (1990) indicated that for “young” clays the uncertainty associated with K_0 from DMT is

about 20%. Coutinho et al. (2006) confirm this result and shows that the Marchetti (1980) K_0 correlation

gives significantly higher values than the reference values considered in this research (see Fig. 25b). Table 8 shows a summary of the results of a quantitative comparative study between the values of K_0

predicted from in situ tests and the reference provided by laboratory testing for Recife soft clays.

7.5 Stiffness and compressibility

Typical curves of void ratio versus effective vertical stress obtained from oedometer tests are shown in

Figure 26 for the two research sites. It can be seen that the "virgin" portion is not linear. This is consistent

with findings in many other investigations (Coutinho 1976, Coutinho & Lacerda 1987, Mesri & Choi

1985). In this study, the virgin portion curve was simplified by two linear parts to obtain the compression

index C_c (C_{c1} and C_{c2}). Figures 27 and 28 show, for the two research sites, the compressibility parameters: compression

index C_{c1} (first part), swell index C_s and recompression index C_r (RRS1). It can be observed that

the deposits are very compressible, particularly the first layer, and that RRS2 presents higher values.

e_0 , C_c and C_s results from Sherbrooke samples are similar to those obtained using Shelby 100 mm

samples. However, C_r results from Sherbrooke samples are much smaller, showing them to be more

representative. The average compression index in layer 1 (RRS1 - $C_{c1} \sim 1.55$; RRS2 - $C_{c1} \sim 2.40$), is

about twice the average values observed in layer 2 (RRS1 - $C_{c1} \sim 0.75$; RRS2 - $C_{c1} \sim 1.2$). In relation

with the difference between the inclination of the two parts of the virgin curve, the results obtained were:

RRS1 (layer 1): $C_{c1} / C_{c2} = 2.08 \pm 0.41$ and RRS1 (layer 2): $C_{c1} / C_{c2} = 1.15 \pm 0.13$; RRS2 (layer 1):

$C_{c1} / C_{c2} = 1.66 \pm 0.21$ and RRS2 (layer 2): $C_{c1} / C_{c2} = 1.52 \pm 0.30$, showing an important non-linearity

in the curves ($e - \log \sigma'_v$) at both research sites. Constrained tangent modulus values (M) from laboratory oedometer tests and DMT tests are compared

in Figures 27 and 28 for the same in situ overburden stress. The Marchetti (1980) correlation for clays

($I_D < 0.6$) was used: The results show a very reasonable agreement in the soft layer 2 of RRS1. In the other layers (RRS2 -

layer 1 and 2; RRS1 - layer 1), in general, M DMT values were slightly higher ($\leq 20\%$) than those obtained

from oedometer tests, M OED. Lunne et al. (1989) stated that, for clays, it is recommended to use the

Marchetti (1980) correlation. Marchetti (1997) concluded that comparisons of M DMT - M lab. or in terms

of predicted vs measured settlements are very encouraging for the use of the correlation M-E D .

Figure 27. Compressibility parameters - oedometer tests and DMT - RRS1 (Coutinho & Oliveira 1997, Oliveira

2002).

Figure 28. Compressibility parameters - oedometer tests and DMT - RRS2 (Coutinho et al. 1999). Statistical correlations (C_c vs. e_0 and C_c vs. w (%)) for the Recife soft clays have been developed

using all data from the laboratory tests database. The results are shown in Figures 29 (a and b). Results

of the Juturnaíba organic soils (Coutinho 1986, Coutinho & Lacerda 1987) are also shown. Due to the

quality of some samples for the Recife organic soils, C_c show more dispersion and probably smaller

values, which can present a slight higher inclination for the correlation.

Figure 29. Statistical correlation between (a) C_c and e_0 (Coutinho et al. 1998b), (b) C_c and w (%). The C_c vs. e_0 and C_c vs. w (%) correlations (Figs 29 a and b) are quite similar for both deposits,

particularly for clay soils. In general, correlations for clay present higher correlation coefficients (r^2) and lower standard error

(lower dispersion) than those for organic soil/peat. The organic soft soils of Juturnaíba research site

present higher correlation coefficients than the Recife deposits, as expected due to the better quality of

samples. The behavior of Recife clays (RRS1) under one-dimensional consolidation is heavily affected by

sampling disturbance (see Table 1, Item 6). The general equation between C_c and w (%) obtained for Recife clays (Fig. 29) is very similar to

the equation presented by Bowles (1979) for organic silts and clays ($C_c = 0.0115 w$), and it is also

very similar to those presented by Djoenaidi (1985) (see Kulhawy & Mayne 1990). Other empirical

correlations that have been suggested, such as, $C_c = w(\%)/100$ for soils with organic content smaller

than 60% (Perrin 1974) and $C_c = 0.010$ to $0.015w\%$, for organic soils, including peats (MPM 1958, from

Ladd 1973) are also reasonable for the Juturnaíba and Recife sites. Futai (1999) presented correlations

for Rio de Janeiro soft soil deposits. For $w \leq 200\%$, the obtained result ($C_c = 0.0108w + 0.2484$) is very

similar to those obtained for the Recife clays equation. Leroueil & Hight (2003) show that for microstructured soils C_c could be approximately defined as a

function of void ratio and strength sensitivity (S_t), as indicated in Figure 30a for the range of consolidation

pressures between σ'_{p0} and $1.4 \sigma'_{p0}$. Figure 30b shows the $C_c - e_0$ relationships obtained for Recife and Juturnaíba organic clays which

have void ratios varying from 0.5 to at least 10 and 1.2 to 9, respectively. In comparison with Figure 15a,

this relationship would correspond to sensitivity of 3 to 6 (Juturnaíba deposit) and of around 8 (Recife

clay deposit), which are reasonable for these deposits, particularly for the Recife clays.

7.6 Vertical and radial consolidation

Conventional oedometer tests were carried with vertical and radial drainage (outflow type) in order

to obtain coefficient of vertical and horizontal consolidation (c_v , c_h) values. For tests carried out with

radial drainage with outflow, c_h values were computed using the Scott (1961) solution, with consideration

“equation strain”. Figures 32 and 33 show the coefficient of vertical consolidation (c_v) versus vertical effective stress

from oedometer tests, for RRS1 and RRS2, respectively. Typical behavior is observed with a very high c_v

values in the overconsolidation range and a rapid decrease around the preconsolidation stress and a value

practically constant in the order of $1.0 \times 10^{-8} \text{ m}^2/\text{s}$ in the normal consolidation range. Figure 33 shows

c_v results for RRS1 obtained from different types of samples. In general the c_v results from Sherbrooke

sample were similar to that from Shelby 100 mm. However, the c_v results from Shelby 60 mm were much

smaller due to the sampling disturbance. Figure 33a shows the bands of values of c_v and c_h for RRS1 obtained from vertical and radial

laboratory oedometer tests. The c_h value was always larger with c_h/c_v in the normally consolidated range

Figure 30. Correlations between C_c and e_0 : (a) clays in general (from Leroueil et al. 1983, Leroueil & Hight 2003);

(b) Juturnaíba organic clays (Coutinho & Lacerda 1987).

Figure 31. c_v (square root method) vs σ'_v (Coutinho et al. 1998c).

lying between 1.0 and 2.5, which is typical for very soft clays (Coutinho 1976, 1980; Ferreira et al. 1986).

Similar results for c_h / c_v (1.0 - 2.0) were obtained by Coutinho (1976) for Rio de Janeiro Sarapuí clay.

Piezocone dissipation tests were also performed to obtain in situ c_h values. The method of calculation

used was that proposed by Houlsby & Teh (1988), with the consideration of initial value u , proposed by

Soares (1986). Figure 33b shows the results obtained (RRS1) and it is possible to see that the piezocone

c_h values (in the overconsolidation range) are much higher than the laboratory test results and of the

order of at least twice. In RRS2, values of c_h were also derived from DMT dissipation tests using the

DMT-C method and the values were higher than c_v obtained from laboratory oedometer tests ($c_h / c_v = 1$

to 3), as expected (see Coutinho et al. 2006).

Figure 32. Coefficient of vertical consolidation vs. average vertical effective stress - Shelby samples 100, 60 mm

and Sherbrooke - RRS1 (Oliveira 2002).

Figure 33. (a) Coefficient of vertical and horizontal consolidation - laboratory vs. in situ test; (b) c_v , c_h and c_h / c_v

vs $\log \sigma'_{vc}$ (Coutinho & Oliveira 1997). RRS1.

7.7 Undrained shear strength

7.7.1 Laboratory

The undrained shear strength (S_u) of Recife clay deposits was extensively studied by means of both

laboratory and in situ tests. The S_u LAB was obtained using good quality samples and by performing UUC and CIUC triaxial tests

(with $\sigma'_c \sim \sigma'_{oct}$ in situ). Figure 34 shows the undrained shear strength (S_u) profile of the two research sites. 0 5 10 15 20 25 10 20 30 40 50 60 70 100 20 30 40 50 100 20 30 40 50 100 20 30 40 50 60 70 S_u (kPa)

D

e p

t h

(m

) UU-C (Oliveira, 1991) CIU-C (Oliveira, 1991) 49.2±4.5
42.0±5.4 35.0±1.5 0 5 10 15 20 25 Su (kPa) Su vane
(Oliveira, 2000) Su disturbed (Oliveira, 2000) S U =
3.50Z+13.51 S U = 1.91Z+22.38 S U = 2.08Z+2.81 0 5 10 15
20 Su (kPa) S U = -2.09Z+34.12 S U = 17.41 S U = e (1.42
+ 0.11 Z) 0 5 10 15 20 Su (kPa) D e p t h (m) S U =
-1.56Z+25.46 S U = 12.99 S U = (e (1.42 + 0.11 Z))/1.09
RRS1 RRS2 UU-C (Oliveira, 2000) CIU-C (Oliveira, 2000) Su
vane (Oliveira, 2000) Su disturbed (Oliveira, 2000)

Figure 34. S U vs. depth (a) Triaxial compression and Field
Vane tests - RRS1; (b) Triaxial compression and Field

Vane tests - RRS2. In RRS1 Su measured in the laboratory
shows values in the range of 30 to 55 kPa, with an average
of

42.0 kPa in the first layer and a higher value of 49.2 kPa
in the second layer (16-26 m). In RRS2, the undrained shear
strength measured in the laboratory shows values in the
range of 7 to

32 kPa, with two different layers, initially decreasing
with depth and afterwards being constant in the

first layer, and increasing with depth in the second layer.
This behavior is similar to that observed for the

Su from field vane test (Fig. 34b).

The two types of laboratory triaxial compression tests
performed in this study (UU and CIU -

$\sigma'_{c \sim \sigma'_{oct}}$ in situ) presented similar values of Su at
both research sites (Fig. 35). These results, in com

parison to the Su obtained using vane test, for the RRS1
presented some difference between each layer,

but practically the same overall average value, while for
RRS2 the Su LAB values were the order of 34%

and 9% smaller in the first and second layer, respectively.

The range of S_u values considering both research sites is representative of all sites investigated in the

Recife Database.

7.7.2 Field vane

Figure 34 shows the S_u values obtained by field vane at the two Recife sites. In RRS1 (Fig. 34a), it can

be seen that the undrained shear strength measured shows values in the range of 34-56 kPa, with three

different intervals and linear increases with depth in each one. In RRS2 (Fig. 35b), the undrained shear

strength values are in the range of 14-37 kPa initially decreasing with depth and then being constant in

the first layer; then increasing with depth in the second layer. Table 5 shows a summary of S_u values

including another Recife soft clay deposit with low S_u results (14-20 kPa). Coutinho et al. (2000b)

present a general discussion about field vane test and present results of undrained shear strength of some

Brazilian soft clay deposits. In general, the S_u values are in the range of 5 to 30 kPa with the exception

of Recife research site 1 (RRS1) deposit, which presents the highest results (34 to 56 kPa). Results of the ratio S_{uv}/σ'_{vp} versus plasticity index from the RRS1 and RRS2 deposits and from other

Brazilian clays are presented in Figure 35. It can be seen that, in general, the Brazilian soft clays

with $I_p < 100\%$ are in agreement with correlations proposed in the literature. If all soils are considered,

including organic soils, the Skempton (1957) correlation presents the best results. In more organic clays

higher strength ratios have been generally obtained with larger dispersion in the results (Coutinho 1986,

Leroueil & Jamiolkowski 1991, Perret et al. 1995). Oliveira (2002) found the following adjusted linear

relationship for the Brazilian clays:

7.7.3 Remolded shear strength

Typical remoulded shear strength (S_{ur}) data obtained by field vane tests are shown for both research sites

on Figure 34.

Figure 35. S_u VANE / $\sigma' p$ vs. I_p correlation (Skempton 1957, Bjerrum 1973) including some Brazilian clays values

(Oliveira 2002). In RRS1, S_{ur} values are in the range of 3.0-10.4 kPa showing an increase with depth in the first layer

and being practically constant in the second layer ($S_{ur} \approx 5.5$). In RRS2, S_{ur} values are in the range of 1.6-4.0 kPa showing an increase with depth in the first layer

and being practically constant with slightly higher values in the first layer ($S_{ur} \approx 2.7$). It has been recognized for some time that a relationship exists between remolded shear strength and

liquidity index. Leroueil et al. (1983) (see also Leroueil & Hight 2003) suggested the relationship: For Recife soft clays this relationship is reasonably consistent, presenting S_{ur} values in the order

of $\pm 30\%$ of the results obtained in this study. For Recife clays, sensitivity has been measured using the in situ vane test and the values obtained

for the research sites are shown on Figure 36 and Table 5. In RRS1 - layer 1 (Fig. 36a), it can be seen

that sensitivity measured shows values in the range of 4.5 to 12, decreasing with depth. In layer 2, the

sensitivity values increase with depth in the range of 9 to 16. In RRS2 - layer 1 (Fig. 36b), the sensitivity

measured is practically constant with depth, showing values in the range of 4.5 to 8. In layer 2, the

sensitivity values increase with depth in the range of 8 to 14. These results suggest that the material can

be classified as “very sensitive”. It is worth mentioning that the sensitivity on the basis of the fall-cone penetrometer is generally larger

than the value obtained from field tests, probably because full remolding cannot be obtained with this

latter test (Leroueil & Hight 2003). Lunne et al. (2003) also found that strength sensitivity can vary

depending on the technique used to measure it. However, for the Onsoy clay the sensitivity was slightly

higher using the in situ vane (6 to 8) than using the fall-cone (4 to 6).

7.7.4 CPTU, DMT, PMT

The undrained shear strength S_u was also obtained from piezocone, dilatometer and pressuremeter in

situ test results at the research sites. The empirical correlations that were used are referenced to the

triaxial compression test or to the field vane test, depending on the case. 0 5 10 15 20 25 0 10 20 Sensitivity D e p t h (m) (a) 0 5 10 15 20 0 10 20 Sensitivity D e p t h (m) (b)

Figure 36. Sensitivity vs. depth. (a) RRS1; (b) RRS2. (Oliveira 2000).

Figure 37. S_u vs. depth: (a) PMT, CPTU, and triaxial compression tests; (b) DMT, uncorrected field vane tests,

and triaxial compression tests; (c) DMT, CPTU, and uncorrected field vane tests - RRS1. Figures 37 and 38 present all S_u profiles for both sites obtained from laboratory triaxial, field vane

and other in situ tests: dilatometer, piezocone and pressuremeter. The results will be discussed for each

type of in situ test.

(a) Dilatometer

The interpretation of dilatometer test results was done in this study using the empirical correlations

proposed by Marchetti (1980), Lacasse & Lunne (1988) and Kamei & Iwasaki (1995):

Figure 38. S_u vs. depth: (a) DMT, CPTU, and uncorrected field vane tests; (b) DMT, uncorrected field vane tests,

and triaxial compression tests; (c) PMT, CPTU, and triaxial compression tests – RRS2 (Coutinho et al. 1999). 0,10 1,00 10,00 1 10 K D 1 10 K D S_u / σ'_{vo} (1) Lacasse & Lunne (1988) (2) Marchetti (1980) (3) Kamei & Iwasaki (1995) RRS1 (Vane) RRS2 (Vane) (1) Lacasse & Lunne (1988) (2) Marchetti (1980) (3) Kamei & Iwasaki (1995) RRS1 (Lab) RRS2 (Lab) Vane 0,10 1,00 10,00 S_u / σ'_{vo} LAB (2) (1) (3) (2) (1) (3)

Figure 39. S_u vs. K D parameters: (a) Vane; (b) Laboratory (Coutinho et al. 2006). Coutinho et al. (2006) commented that at RRS1 S_u values obtained by Marchetti's correlation in

general are close to or slightly higher than the vane tests and the laboratory triaxial results. The S_u values

from the Lacasse & Lunne (1988) correlation were in general close to the laboratory triaxial tests and

lower or close to the vane tests results (Figs 39 a and b). In RRS2 the S_u values from Marchetti's correlation are in general close to or slightly lower than the

vane tests and close or slightly higher than the laboratory tests results. The S_u values from the Lacasse &

Lunne (1988) correlation were close to or slightly lower than the triaxial compression tests and lower

than the vane tests results (Figs 40a, b). Considering both research sites, it can be observed that for the Recife soft clay the estimation of S_u can

be obtained with reasonable confidence for practical purposes, using the original correlation (Marchetti

1980) for vane test and the correlation proposed by Lacasse & Lunne (1988) for triaxial compression test

results. Table 8 shows the results of a quantitative

comparative study between the values of S_u predicted from in situ tests and reference test results.

(b) Piezocone

The tip resistance is related to the undrained shear strength by the cone factor:

Figure 40. Strength envelope- CIUC triaxial tests (RRS1). Despite theoretical advances, empirical correlation is still the most commonly method used for the

interpretation of piezocone tests. In this study, the empirical correlation used were: Tavenas & Leroueil

(1987) - N_{KT} vs. IP (field vane tests), and the correlation proposed by Lunne et al. (1985) - N_{KT} vs B_q

(compression triaxial tests). In general, the results obtained are in agreement with other studies (Lunne et al. 1989) confirming the

possibility of good prediction of S_u from the piezocone tests. Table 7 shows a quantitative comparative

results, showing that the use of $N_{KT} = 12 \pm 1.3$ in the Tavenas & Leroueil (1987) correlation is an

appropriate value.

(c) Pressuremeter

The undrained shear strength was also obtained from pressuremeter tests using the Powell (1990)

correlation: In general, the results obtained are in agreement with the laboratory S_u values for both Recife sites,

confirming the possibility of good prediction of S_u . Table 7 shows results of a quantitative comparative

study between the values of S_u predicted from in situ tests and reference results, showing that the use of

the Powell (1990) correlation is an appropriate prediction.

7.8 Drained strength

The laboratory drained shear strength parameters were obtained, for the research sites, on the basis of

isotropically consolidated undrained compression (CIUC) triaxial tests, with pore-pressure measure

ments, using Wykeham Farrance type equipment, with specimens of 37.5 mm in diameter and 80 mm

long or $d = 50.6$ mm and $H = 105$ mm. Figures 40 and 41 show, for the two Recife clay sites, the effective strength envelopes obtained from

points corresponding to peak results. The curvature of the envelope in the overconsolidated range can

be clearly observed, showing higher values than in the normally consolidated range.

Table 5 summarizes the effective friction angles in the normally consolidated range (ϕ'_{NA}) using

laboratory triaxial and in situ piezocone tests.

The average values obtained of ϕ'_{NA} are in the range of 25-26 ° (RRS1 - Layer 1), 23-28 ° (RRS1 -

Layer 2) and 25 ° -27 ° for RRS2. The Lunne et al. (1985) proposal using CPTU results showed values

relatively reasonable to the ϕ'_{NA} for RRS1.

7.9 Yield surface

A yield curve for specimens of RRS1 - Layer 1 (8.0-16.0 m) is shown in Figure 42. Although the data

are somewhat limited, it can be seen that the limit state curve (LSC) is not centred on the isotropic axis 0 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150 160 $p' = (\sigma'_1 + \sigma'_3) / 2$ (kPa) $q = (\sigma'_1 - \sigma'_3) / 2$ (kPa) $\alpha' = 22,8^\circ$ CIU (OLIVEIRA, 2000) CIU (SOARES, 2005) $q = 0,42 p' \pm 0,04$; $r^2 = 0,94$; $\phi' = 24,8^\circ$

Figure 41. Strength envelope - CIUC triaxial tests - RRS2 - Layer 1 (4 to 11.5 m).

Table 5. Effective friction angle in the normally consolidated range Laboratory and in Situ tests (from Coutinho

et al. 1993) ϕ (°) Piezocone ϕ ' (°) Triaxial CIU-C
Senneset & Lunne Coutinho Amorim Oliveira

Site Layer Depth (m) Janbu (1984) et al. (1985) et al.
(1993) Jr. (1975) (2000)

RRS1 I 7-16 25-33 18-23 26 25 - II 16-26 31-33 24-27 23 28 -

RRS2 I - - - - 25 II - - - - 27

as has been observed for natural clays, due to their
anisotropic properties (Diaz-Rodriguez et al. 1992,

Almeida & Marques 2003). It is possible also to see that
the effect of 60 mm diameter Shelby sampling is to
destructure the clay

and “shrink” the yield surface (Hight et al. 1992, see also
Lunne et al. 2003 and Leroueil & Hight 2003).

7.10 Deformation data

Failure of Recife soft clays is relatively brittle
presenting a peak, and strain at failure varies in general

from 1 to 3% (Fig. 43). Figure 44a shows values of the
Young modulus, E_u , obtained at 50% of stress

at failure, from triaxial UUC for RRS1. A great dispersion
can be observed with an average value of

11.5 MPa for the first layer and 12.6 for the second layer.
The average E_u / σ_u relationship seems to be

constant for each layer, $E_u / \sigma_u = 251$ (Layer 1) and $E_u / \sigma_u = 285$ (Layer 2).

7.11 Viscous behavior

Conventional oedometer tests have been carried out to
determine creep rates by the inclination of the

initial linear portion of the time-settlement curves.
Results of $C_{\alpha\epsilon} (\sigma_e / \log t)$ for both research sites are

shown in Figure 45. Values of $C_{\alpha\epsilon}$ were normally small for
low stress, rapidly increases around the pre-consolidation

stress, reaching a maximum for a stress greater than σ'_p

(1.5 to $2.5\sigma'_{p}$), and then decrease. Coutinho

(1976, 1986) found similar behavior for other Brazilian clays (Sarapuı́ - RJ and Juturnaı́ba-RJ). The C_{α}

values under one-dimensional consolidation are heavily affected by sampling disturbance; increasing

in the overconsolidation range and reducing in the normally consolidation range (see Coutinho 1976,

Coutinho et al. 1998a). These results are in agreement with the reported behavior of clays (Ladd 1973). Table 6 shows results of $C_{\alpha max}$, O.C. and $C_{\alpha max} / CR_{max}$ obtained on Recife and other Brazilian organic

clays. It can be seen that in general, soils with higher O.C. show higher $C_{\alpha max}$ and $C_{\alpha max} / CR_{max}$ (both

Figure 42. Yield curve for Recife soft clays (Oliveira 2005) (sampling Shelby 60 mm - Amorim Jr. 1975, Shelby

100 mm - Coutinho et al. 1993; Sherbrooke - Oliveira et al. 2000).

Figure 43. Deviatoric stress-axial strain relationships - UUC triaxial test - RRS1.

obtained for same range of stress just after σ'_{p}). The range found is in accordance with that proposal by

Mesri & Choi (1985) of 0.05 ± 0.01 for organic for organic clays and of 0.04 ± 0.01 for inorganic clays.

Very organic soils / peats can present higher values. Undrained shear strength typically changes 5 to 20% per logarithm cycle of strain rate for soft clays

(Leroueil & Marques 1996). Results for Recife clays have not yet been measured in the laboratory, but

evidence of strain rate effect on normalized S_u of Juturnaı́ba-RJ organic clay was presented by Coutinho

(1986), where an average change of 10.5% per logarithm cycle of strain rate in CIU tests was measured

on normally consolidated samples as shown in Figure 46.

7.12 The influence of the organic content (O.C.)

The influence of the organic content (O.C.) on soils properties is primarily a function of its type, amount, and degree of decomposition. The natural water content, the plastic and liquid limits, and the plasticity index of organic clays tend to be higher than those of the inorganic clays of the same mineralogy, due to the ability of the organic matter - clay matrix to absorb water. On the other hand, the density of soils tends to be smaller, due to this ability and also to the lower density of the organic matter (Coutinho & Lacerda 1987, Mitchell 1993). The compression index increases with increasing liquid limit and natural water content or void ratio and strength sensitivity (Kullawy & Mayne 1990, Mitchell 1993, Leroueil & Hight 2003). As the presence of organic matter further increases these index parameters, it will also serve to increase the soil compressibility. The value of coefficient of secondary compression ($C_{\alpha\epsilon}$) also increases

$C_{\alpha\epsilon}$	0	4	8	12	16	20	24	28	0	5	10	15	20	25	30	Eu (MPa)	Depth (m)
	11.5	±6.0	12.6	±4.6	0 <td>4 <td>8 <td>12 <td>16 <td>20 <td>24 <td>28 <td>0 <td>200</td> <td>400</td> <td>600</td> <td>Eu/Su</td> </td></td></td></td></td></td></td></td>	4 <td>8 <td>12 <td>16 <td>20 <td>24 <td>28 <td>0 <td>200</td> <td>400</td> <td>600</td> <td>Eu/Su</td> </td></td></td></td></td></td></td>	8 <td>12 <td>16 <td>20 <td>24 <td>28 <td>0 <td>200</td> <td>400</td> <td>600</td> <td>Eu/Su</td> </td></td></td></td></td></td>	12 <td>16 <td>20 <td>24 <td>28 <td>0 <td>200</td> <td>400</td> <td>600</td> <td>Eu/Su</td> </td></td></td></td></td>	16 <td>20 <td>24 <td>28 <td>0 <td>200</td> <td>400</td> <td>600</td> <td>Eu/Su</td> </td></td></td></td>	20 <td>24 <td>28 <td>0 <td>200</td> <td>400</td> <td>600</td> <td>Eu/Su</td> </td></td></td>	24 <td>28 <td>0 <td>200</td> <td>400</td> <td>600</td> <td>Eu/Su</td> </td></td>	28 <td>0 <td>200</td> <td>400</td> <td>600</td> <td>Eu/Su</td> </td>	0 <td>200</td> <td>400</td> <td>600</td> <td>Eu/Su</td>	200	400	600	Eu/Su
	Depth (m) 251.3±108.1 285.7±120.3																

Figure 44. Eu and Eu/Su vs. Depth - RRS1.

Figure 45. $C_{\alpha\epsilon}$ vs. σ'_{v0} (a) RRS1 (Coutinho & Ferreira 1988), (b) RRS2 (Oliveira 2000).

with increasing natural content; therefore, soft organic clays have relatively high values of $C_{\alpha\epsilon}$ and the ratio $C_{\alpha\epsilon}/CR$ (Coutinho & Lacerda 1987, Mesri & Godlewski 1977). The amount and the type of the organic matter and its relationship to the solid particles in the soil are also very important in influencing the stress-deformation-strength behavior. In soft organic clays the ratio of the undrained shear strength to the preconsolidation pressure S_u/σ'_p has usually been observed to be higher and with more scatter

than for inorganic clays. The overconsolidation due to secondary compression should be also higher

in sediments with significant organic matter (Larsson 1980, Leroueil et al. 1990, Coutinho & Lacerda

1989, Perret et al. 1995). Perret et al. (1995) examined various processes controlling the development

of strength in fine-grained sediments of the Saguenay Fjord, Quebec. In particular, both the role of

organic matter (OM) and the mode of deposition of sediment were evaluated as it relates to the origin

Table 6. $C_{\alpha max}$ and $C_{\alpha max}/C_{R max}$ values for some Brazilian soft clays (Coutinho 1988). $C_{\alpha max}$ (%) $C_{\alpha max} / C_{R max}$ Layer or O.C. (%)

Deposit Depth (m) (average) Range Average Range Average

Recife - RRS1 10 - 16 6 1.34 to 2.28 1.67 0.043 to 0.053 0.047

Recife - RRS2 4 - 11.5 7 2.35 to 3.3 2.75 - -

Juturnaíba Ib 18 0.84 to 1.2 1.02 0.034 to 0.038 0.036

(Rio de Janeiro) II 43 1.92 to 2.84 2.35 0.040 to 0.067 0.050 III 25 1.80 to 2.85 2.36 0.048 to 0.071 0.060 IV 60 1.84 to 2.84 2.40 0.043 to 0.065 0.055 VI 1 0.33 to 0.58 0.42 0.028 to 0.042 0.032

Sarapuí 5-7 5 1.81 to 3.51 2.30 0.048 to 0.082 0.056

(Rio de Janeiro) 1.81 to 2.33* 2.10 0.048 to 0.055* 0.052

*Values without the one only specimen that showed very high $C_{\alpha max}$ (3.51%).

Figure 46. Influence of strain rate effect on S_u - Juturnaíba - RJ (Coutinho 1988).

of overconsolidation. Figure 47a shows results of the $S_u / \sigma'_{p, Ip}$ and OM relationships obtained, showing

for these deposits a more dependant on the organic matter content than the soil plasticity. Figure 47b shows the influence of organic content on some geotechnical

properties in Juturnaíba soft

organic soil in Rio de Janeiro (Coutinho & Lacerda 1987, 1989). See also Mitchell & Coutinho 1991.

7.13 Comparative study - reference test vs. in situ tests correlations

Table 7 shows a summary of the correlations used in the present research to obtain OCR, K_0 , S_u

and M from in situ tests results. Values of the geotechnical parameters obtained were compared with

correspondent reference values (OCR - oedometer test, $K_0 = (1 - \sin \phi')$ OCR $\sin \phi'$, S_u - triaxial and vane

tests and M - oedometer test). Column 5 of Table 7 (Recife Experience) shows the results from the quantitative comparative study

between the geotechnical parameters values predicted from the in situ tests and the results from the

reference values. It can be observed that for the Recife soft clay the estimation of geotechnical parameters

is quite accurate for practical purpose from results of in situ tests, using correlations from the literature.

Column 6 presents the most recommended in situ tests correlations to be used in the Recife soft clays

deposits, showing also the uncertainty associated with the prediction of the geotechnical parameters.

Figure 47. (a) Relationship between normalization ratio C_u/σ'_{p_0} , plasticity index, and organic matter (OM) percentage

(Perret et al. 1995); (b) Effect of organic content on compressibility and strength properties of Juturnaíba soft organic

nical parameters of a clay deposit, but in general they are not accurate enough to determine a parameter

profile that is usable for final designs. Consequently, in Recife clay deposits, as it has been observed

in many studies, the in situ tests must be used to interpolate or extrapolate on the basis of geotechnical

parameter values previously determined in reference procedure - tests or to define approximate values

that can be used for preliminary designs.

8 PRACTICAL APPLICATIONS

8.1 Steel pile under lateral loading in a very soft clay deposit

In 1995, a complete rupture in a reinforced concrete structure of a floor supported on steel piles embedded

in a 17 meter thick soft clay layer in Recife, Brazil, occurred 21 years after it had been built, with no

warning of potential failure. Figure 48 presents the geotechnical profile of a cross section of the area and the hypothesis proposed

for the accident. A slow lateral movement of the organic clay layer provoked lateral displacement of

the piles which were supporting the total vertical load (structure self weight + negative friction) causing

a buckling failure. This case demonstrates the importance of a buckling study in steel piles caused by

lateral displacement in soft soil. Afterwards, the Geotechnical Group of the Federal University of Pernambuco, Brazil, performed

an extensive geotechnical research program in the area (UFPE - RRS2). A study was developed on the

behavior of laterally loaded steel piles in thick layers of soft clay, consisting of analytical and experimental

stages (Braga 1998, Coutinho et al. 2005). In the experimental stage, lateral loading tests in steel piles

driven into the organic clay deposit were carried out where the aforementioned accident had taken place. In the analytical stage, predictions of the horizontal displacements of the pile's top and also for the

buckling load of a steel pile in very soft clay were made from linear and non-linear analyses using the

finite element method. The soil was modeled with p-y curves obtained from dilatometer (DMT) and Ménard pressuremeter

(PMT) test results performed at the site of the accident and near the damaged structure. The following

assumptions were considered: the steel pile was perfectly vertical and had vertical load eccentricity,

that is, with initial lateral deformation. The p-y curves were found through the semi-empirical method

proposed by Robertson et al. (1989), which uses data from dilatometer tests, and for the semi-empirical

method proposed by Ménard et al. (1969), which uses data from pressuremeter tests. The horizontal displacements were measured (inclinometer) and predicted with linear and nonlinear

FEM analyses for level land grades and after excavation of the fill. Figure 49 and Table 8 present the

results obtained versus the applied loads. It can be noted that the nonlinear analyses (DMT and PMT)

results are very close to the values measured, showing differences ranging from 1% to 20%. In the analysis for the collapse of the steel piles, two important facts must be taken into consideration:

(a) whether the steel pile was completely vertical and; (b) whether there was any eccentricity in the

vertical load. It was assumed that the steel pile suffered horizontal displacements and showed a second degree

parabolic form. These displacements were induced by lateral nodal loads to produce values of 10, 20,

30, 40, 50, and 100 mm at $L/2$. The analysis of the results of critical loading due to accidental displacement

performed according to ANSYS (1989) is summarized in Table 9. It can be observed that the critical

load is very sensitive to the effect of accidental

displacement which rapidly decreases its value. The loading capacity of the steel pile under analysis was calculated through the Aoki-Velloso method

(1975) using data from SPT performed at the accident site. As shown in Table 9 the working load for the

steel pile would be 186.5 kN and was within the interval which determines the occurrence of failure of

displacement between 30 and 50 mm.

8.2 Analysis of static load tests of precast - instrumented concrete piles in a soft clay deposit

At RRS2, research work was developed with the principal objective of making a detailed study of pile

foundation, through instrumented static vertical load tests of precast-concrete piles, floated in the soft

clay deposit. It was also investigated if the increase of resistance with time in clay - the called "set-up"

effect - could be considered in foundation projects in Recife. This effect can provide in some cases

appreciable economy due to the thickness of the soft clays deposits (Soares 2006, Soares et al. 2006).

Figure 48. Geotechnical profile - horizontal pull (Coutinho et al. 2005). Table 8. Predicted and measured displacements (Braga 1998, Coutinho et al. 2005) H = (kN) Horizontal displacements (mm) 2.5 5.0 7.5 10.0 Linear analysis 25.95 51.9 77.85 103.81 DMT 9.22 27.09 56.85 108.56 PMT 10.32 33.52 63.27 106.74 Measured 18.59 27.86 68.71 109.65 0 2 4 6 8 10 12 0 20 40 60 80 100 120 Displacement (mm) L o a d (k N) Measured (Inclinometer) Predicted (DMT) Predicted (PMT) Linear Analysis Medium Curve (measured)

Figure 49. Predicted and measured displacements (Braga 1998, Coutinho et al. 2005). For the study 4 centrifugal pile tests (EC350 - D = 350 mm/L = 9.70 m) and 12 prestressed precast

concrete reaction piles (EPH280 - D equiv = 280 mm/L = 9 m) were driven. Figure 50 shows the location

of the piles (test and reaction), the system of reaction used in the static load tests and the soil profile of

the experimental field (RRS2). Figure 51 shows the results of 10 maintained vertical loading tests carried out on the piles E1 and E2.

The total load (bearing capacity) and also the residual strength initially reduce to one definitive limit,

from which the load / strength starts to grow with time to a value higher than the first ultimate strength.

It was also observed that increasing the interval between two subsequent load tests can give a significant Table 9. Critical loading due to accidental displacements (Braga 1998, Coutinho et al. 2005) Critical Loading (kN)
Displacement Curves P-Y (DMT) Curves P-Y (PMT) (mm)
Free/Labeled top Free/Labeled top 0 2988.64 1925.11 10
1738.18 1877.44 20 1183.99 510.21 30 360.29 98.86 50 58.19
70.89 100 46.25 56.90

Figure 50. (a) Location of the piles and system of reaction; (b) geotechnical profile of the experimental field site

(Soares 2006).

gain of resistance. This behavior presents some differences to that usually described in the literature,

which has shown a permanent growth of the ultimate strength with time (set up effect). The process of driving piles in clay can strongly disturb the soil and probably increases pore-pressures

during driving. At the site investigated, the deposit is 18 m thick, presents a low permeability and the

soft clay has a very high sensitivity (see Table 4). These conditions increase the disturbance in the soil

and the time to dissipate the excess pore-pressure generated. It was also interesting to observe that pore-pressure measurements u_1 and u_2 in piezocone tests showed

pore-pressure redistribution, between the u_1 and u_2 locations, just at the beginning of the dissipation tests. Figure 52 shows measured (from pile load test) and predicted values of the bearing capacity of the

driven concrete piles, using theoretical and semi-empirical methods. The methods of prediction were

based on the geotechnical parameters of the soft clay obtained from laboratory and in situ tests and

have been presented in this paper. A great variation of predicted results can be observed. The theoretical

method that presents the best results was the β Method, showing a value of total load 21.3% higher than

the measured one. For the semi-empirical methods, the PMT and DMT based methods give very good

results, showing a difference in the order of 3.4% higher and 6.9% lower, respectively. In relation to the time effect, a reduction in the total load (in relation with the previous test) was

observed up to seven days. However with the increase of time between the tests (for example 40 days),

the total load increased by 11.5% in relation to the ultimate strength and 21.2% in relation to the residual

strength (Figure 51). From results of this study and another experience in Recife, Soares (2006) presents

a procedure to consider the increase of resistance with time in pile foundation projects in Recife clays,

using methods proposed in the literature (see for example Karlsrud et al. 2005-“NGI-99” method). Pile E1

Load (kN)	Settlement (mm)	1 ^o Test-30h	2 ^o Test-3days	3 ^o Test-7days	4 ^o Test-39days	5 ^o Test-71days	Smax
133	124	120	114	146	142	146	142
111	106	146	142	146	142	146	142
146	142	146	142	146	142	146	142

Pile E2

Load (kN)	Settlement (mm)	1 ^o Test-30h	2 ^o Test-3days	3 ^o Test-7days	4 ^o Test-39days	5 ^o Test-71days	Smax
129	111	115	111	105	102	142	133
146	146	146	146	146	146	146	146

Figure 51. Results of maintained loading tests (Soares 2006). 30 30 11 12 11 8 18 78 213 254 146 90 136 30 116 107 104 65 101 131 265 122 73 225 1^o PLT 5^oPLT Met. A-V Met.

D-Q Met. Tex. Met. CPTu Met. PMT Met. DMT Load of Tip
 Lateral Load Total Load 30 30 19 19 19 101 116 192 139 229
 146 211 159 249 131 0 50

100

150

200

250

300 1^o PLT 5^oPLT

L o

a d

(k

N) Load of Tip Lateral Load Total Load Met. α Met. β Met.
 λ (a) Theoretical Methods Reference χ Tomlinson (1957)
 δ Burland (1973) ν Vijayvergiya & Focht (1972) (b)
 Semi-empirical Methods Reference SPT Test Aoki-Velloso
 (1975) Décourt-Quaresma (1978) Teixeira (1972) CPTU Test
 Almeida et al. (1996) PMT Test Clarke (1995) DMT Test
 Powell et al. (2001b)

Figure 52. Comparison of measured and predicted total load:
 (a) based on theoretical methods (b) based on

semi-empirical methods - RRS2 (Soares 2006).

8.3 Settlements of a building with a shallow foundation in
 Recife soft clay

This study is part of joint research between the
 Geotechnical Group - Federal University of Pernambuco,

AMBITEC Group - University of Pernambuco and Company Gusmão
 Consulting Engineers - Recife/PE

(Oliveira et al. 2004). Based on results of an instrumented
 building built on the Recife soft clay deposit, the main
 objective

of this investigation was to evaluate and compare measured
 settlements and those estimated through

Figure 53. (a) Geotechnical profile of the subsoil (Gusmão

Filho et al. 1999), (b) Plan view of foundation with

predicted settlements without soil-structure interaction (Gusmão Filho et al. 1999).

geotechnical parameters from laboratory tests and technical information from the Database of Recife

soft clays (Coutinho et al. 1998a; 2000a). The studied building was described by Gusmão Filho et al. (1999) and it consists of a reinforced

concrete framed structure with 15 stories and 17 columns, whose vertical loads vary between 1510 and

5770 kN. Figure 53a shows the geotechnical profile of the subsoil that is located in the neighborhood of Boa

Viagem Beach. Improvement of the superficial layer of sand was executed using compaction piles: 910

piles composed of a mixture of sand and gravel were installed on a squared grid of 90 cm side over the

total area of the building. The piles had 300 mm diameter and 5 m length. The designed foundations were

footings at 1.5 m depth and allowable bearing pressure of 400 kPa (Fig. 53b). Sampling of the soft clay

deposit (10-25 m depth) was carried out by a private company using Shelby samplers of thin wall with

100 mm diameter and area ratio of 5%. The clay presents low plasticity ($IP = 20-25\%$) and its natural

water content (w) fell between 40 and 48%, while the average liquid limit (w_L) was 54%. The prediction of the settlements was made by conventional procedures based on results of oedometer

tests and without considering soil-structure interaction. Due to some difficulties the quality of the sam

pling was not good (poor to very poor), according to the approach of Lunne et al. (1997). Table 10a shows

the average geotechnical parameters adopted in the original design (Gusmão Filho et al. 1999). Figure

53b shows the predicted settlements in each footing for the final loads, with an average value of 146 mm. To monitor the building performance, settlement plugs were installed in all columns. The reference

measurement was made in June 1996, when only 3 slabs had been constructed. The building was inhabited

in May 1998. Until May 2000, 10 settlement measurements were made during the construction period

and three were made after the building was inhabited. The corresponding average loading was estimated

according to the number of stories executed. Figure 54a presents the evolution of minimum, average and maximum settlements versus time of

loading. The settlements show the beginning of a tendency to stabilize (the last three measurements)

about 8 months after the end of the construction of the building. Figure 54b shows the evolution of the average settlement rate versus time of loading. It is observed

that there was a tendency of growth of the values in the construction period and then the settlement rate

diminished in the last 14 months. The average rate in the last measurement was of 36µm/day. If a linear

Table 10. (a) Parameters of compressibility of the soft clay adopted in the design (Gusmão Filho et al. 1999); (b)

Relation between measured and predicted settlements - 13th measurement (a) (b)

Depth σ'_{vo} Measured settlement /

(m) (kPa) e_0 C C C S OCR Column Predicted settlement

10-12 110 1.23 0.56 0.05 1.20 P4 0.84

12-14 125 1.10 0.26 - 1.00 P5 0.79

14-18 143 1.00 0.16 - 1.00 P8 0.84

18-25 176 1.26 0.48 - 1.00 P9 0.53 P10 0.55 P11 0.68 P15 0.76

Notes: σ'_{vo} = vertical effective overburden pressure, e_0 = initial void ratio, C_c = compression index,

C_s = recompression index, OCR = overconsolidation ratio.

Figure 54. (a) Evolution of loading and minimum, average and maximum total settlements; (b) Evolution of

minimum, average and maximum settlement rates.

regression of the rates obtained in the last 4 measurements is accepted then the complete stabilization

of the settlements would happen on November 2001 and the final average settlement would be 105 mm. In conventional structural design the column bases of buildings are assumed to have no displacement.

This consideration influences the computation of loads at the foundation, dimensions of structural beams,

design of the foundation and settlement estimated. When the soil - structure interaction (SSI) effects are

considered, due to the settlements and structural stiffness, it is observed that a redistribution of loads on

structural beams and column and a restriction of relative movements, makes the differential settlements to

be less than those conventionally calculated, although the average settlement is practically the same. All

these effects can make the values of settlement allowable that otherwise would cause damage (Gusmão

Filho et al. 1999).

Table 10b presents the relationship between measured values and those predicted for some columns

for the period of 1000 days (13th measurement). The values of this relationship vary between 0.53 and

0.84, the minimum values referring to the columns at the center and the maximum values to the columns

at the periphery. This observed difference is due to: (i) sample quality; (ii) redistribution of loads due

to the soil-structure interaction; and (iii) some uncertainty in the calculation of the stress distribution in

the soft clay (the effect of improvement of the superficial sand layer). Normalized stress-strain curves (ratio between the settlement and thickness of layer) for several

columns of the analyzed building together with the normalized stress-strain curves obtained in oedometer

tests are shown in Figure 55a. The results can also be presented considering the average normalized

settlement of the building. The idea of this interpretation is to analyze readings of settlement as a

consolidation test in the field, with the limitations in terms of contour conditions, especially the fact that

the drainage may be partial in each stage. A difference between field and laboratory curves is observed.

Figure 55. (a) Stress-strain normalized curves - Building analyzed versus laboratory - Recife - Brazil,

(b) Stress-strain curves - Laboratory and field (Leroueil et al. 1988).

Figure 56. Curves of settlement comparative versus final stress - samples of good, poor quality and using

correlations - Depth 16.50-17.30 m (Silva & Coutinho 1999).

This behavior is similar to that observed by Leroueil et al. (1988) studying embankments on soft soils

(Fig. 55b), where the difference in the curves is considered to be due to the difference in the strain rate

and so there is a value of specific strain that should be added to the laboratory curve to obtain the field

curve. However, in the case studied the yield stress in the field is smaller than in the laboratory. The geotechnical parameters values and consequently calculation of the settlement can be strongly

affected if the quality of the sample is inadequate. To investigate this effect in the Recife soft clays,

a parametric study was accomplished considering samples of good and poor quality collected in the

deposit of SESI-Ibura in Recife (RRS2 - Recife Research Site 2). Normalized settlement (ratio between

the settlement and thickness of layer) vs. applied vertical stress was estimated by Silva & Coutinho (1999)

through an analytical method using three different ways of obtaining the geotechnical parameters: (i) from

samples of good quality; (ii) from samples of poor quality; and (iii) correcting only the compression

index (C_c) of samples of poor quality using the Recife Database statistical correlation (C_c vs. w (%)) -

see Fig. 29). Figure 56 presents results of one of these comparisons. It is observed that: (i) the highest

value of the settlement is in the case of sample of poor quality with only C_c corrected by the correlation;

(ii) the value of the settlement calculated from the sample of poor quality without correction it is always

larger than the value obtained by the sample of good quality. The difference is more significant at stresses below 200 kPa which is the working range for the great

majority of the buildings in Recife. This fact can be one important cause of the difference observed

Figure 57. Difference between the localization of the surface foreseen and observed - 50% cracking of

embankments - S_u without correction (Bello 2004).

Table 11. Summary of the FS min calculated with the GEO SLOPE Program (Bello 2004). No circular surface Circular surface Back-analysis Cracking of Correction embankment (%) of S_u Analysis Back-analysis Janbu Morgens.

Hypothesis 0 50 100 yes no Bishop Spencer Bishop Spencer corrected Spencer -Price

1 X X 1.045 1.048 1.192 1.190 1.240 1.232 1.232

2 X X 1.356 1.357 1.475 1.472 1.480 1.397 1.397

3 X X 1.000 0.995 1.149 1.141 1.128 1.141 1.141

4 X X 1.297 1.290 1.474 1.462 1.481 1.507 1.507

5 X X 0.896 0.899 1.100 1.103 1.037 0.942 0.942

6 X X 1.168 1.168 1.316 1.316 1.244 1.336 1.336

7 X X 1.082 1.076 1.205 1.195 1.153 1.186 1.186

between the estimated and measured values of settlements (Table 10b), since the samples of the project

are classified as poor or very poor.

The relation among the calculated settlement with samples of good and poor quality is in the order

of 0.75 (0.60 to 0.90) for the range of stress between 80 and 200 kPa (Fig. 56). If this relation obtained

for RRS2 can be considered representative for the Recife lowland, the corrected average settlement of

the building studied would be: the average predicted settlement 146 mm $\times$ the average correction factor

0.75 = 110 mm. This value is very close to the final average measured settlement, which was obtained by

extrapolation from the measured settlement (105 mm). This result is very interesting because the average

predicted settlement is practically the same with or without consideration of soil-structure interaction.

This study represents a research line in development and it can be summarized by some points:

(i) the sample quality and the soil-structure interaction are very important for the adequate estimation of

settlements of building on soft clay; (ii) a correction factor (average of 0.75) from the Database of Recife

soft clays is a preliminary proposal to correct the predicted settlement in Recife plain area, to take into

account the effect of the sample quality; (iii) The interpretation readings of settlements of buildings as a

“equivalent” oedometer test in the field is considered to have the potential to improve the understanding

of the foundation behavior; and (iv) the importance of considering the complete geotechnical parameters

(σ'_{p} , C_c , etc) when using a statistical correlation (Database) to correct the values to estimate the predicted

settlement.

8.4 Failure of the embankment on soft soil in Recife - Brazil

Failure of an embankment on soft clays occurred in an area besides the federal road BR-101-PE, Recife,

Brazil. The data was obtained from partnership in the Project PRONEX CNPq/MCT-UFPE, the company

Gusmão EngineersAssociates and the Federal University of Pernambuco, Brazil (Bello 2004; Coutinho &

Bello 2005; Bello et al. 2006). The soil profile at this location was a 12 meter thick layer of soft soil.

The embankment was constructed without geotechnical investigation and control.

Figure 58. Factors of correction from back-analysis of embankment failure (Coutinho et al 2000b, Sandroni 1993,

Massad 1999 and Bello 2004). After failure, an urgent and limited geotechnical investigation was performed to allow an analysis of the

behavior of the embankment and recommendation for the owner. Two SPT soundings with water content

measurement, two vane test soundings, undisturbed sampling (only one sample) and correspondent

laboratory tests - characterization, oedometer and triaxial UU-C was performed. During the research work the Recife soft clays Database of UFPE was used to complement the technical

information necessary for the study, which associated with the water content measurement in the field,

showed itself to be a very good tool to give support to professional and research work. Figure 57 shows the construction, soil profile and some technical information about the site. The

analysis of stability (search for the critical surface) and the back-analysis (from “observed failure sur

face”) were done by limit equilibrium methods using the “Geo Slope Program”, considering circular and

non-circular surfaces, cracking of the embankment and the necessity or not to correct the S_u from vane

tests. The Bjerrum (1973) and the Aas et al. (1986) proposals were considered. Table 11 shows a summary of the results in terms of critical factor of safety (FS_{min}). For the condition

of analysis of stability in circular surface the results of the FS_{min} are in the range of 0.896 to 1.356.

The values of FS_{min} considering the correction of S_u (Bjerrum 1973 proposal) are close to 1.0, showing

the need to correct the vane shear strength for Recife soft clays deposits. The influence of cracks in

the embankment on the values of the FS_{min} was not so significant (reduction of the order of 10–15%).

Due to the shape of the “failure surface”, the three-dimensional effect (Azzouz et al. 1983) showed an

insignificant increase (of the order of 5%) in the critical factor of safety. The back-analysed FS_{min} results presented slightly higher values (8 to 24%) compared to the corre

sponding results from the stability analysis. This difference can be due to the difficulty in defining the

real failure surface in the field, with the information available in the study. The stability study was also done using some empirical methods (Equation of Load Capacity, Method

of the Sliding Wedges, Chart of Pillot & Moreau 1973, Chart of Pinto 1966) and the results were

satisfactory with FS close to 1.0 for failure conditions, using representative corrected values of S_u

(Bjerrum proposal). Figure 58 shows the Bjerrum (1973) correction factor including Brazilian results from five analyses of

embankment failure on soft clay deposits. The case history studied in Recife clays is also shown using the

average plasticity index of the deposit. With the exception of Juturnaíba organic soils deposit all of them

confirm the necessity of the correction of S_u vane to be used projects in the Brazilian organic soft clays.

9 CONCLUSION

Recife clays are a 10,000 year old fluvial/marine deposit. Different types of soft soil profiles can be

observed in the coastal plain area, with the two research sites investigated (RRS1 and RRS2) presenting

about 20 m of thickness, and at least two different layers of geotechnical properties. The deposits studied

are highly plastic clays; I_p is 70% and 33% in RRS1 and 98% and 53% in RRS2, with the higher values

in the first layer. Its average liquidity index is 0.66-0.71 (RRS1) and 0.81-0.90 (RRS2). Its average

strength sensitivity is in the range of 6 to 13, depending on the layer / depth of the research deposit. The

content of particles of clay size is generally in the range of 60 to 70%. The main clay mineral is Kaolinite

with the possible presence of a more active mineral. In general, the organic content is in the range of

3 to 10%, for the Recife organic clays. Organic soils deposits or layers (with O.C. >10%) can also be

found in the Recife coastal plain area. Numerous tests have been performed, in the laboratory and in situ, to examine

sampling quality and

determine the mechanical and “hydraulic” characteristics of the Recife clay deposits. Experiences on Recife clays (at the research sites) show that good quality samples are difficult to

obtain but can be achieved using stationary piston or Shelby sampler with 100 to 115 mm diameter and

extra care in situ, transportation and laboratory. The Sherbrooke block sampler gave similar results or

better quality in comparison with Shelby 100 mm.

The upper part of the deposits is overconsolidated due to aging, desiccation and water table fluctuation,

generally showing values of overconsolidation ratio OCR varying from 2.5 to 1.0. The second layer at

the research sites is practically normally consolidated. Values of OCR (or preconsolidation pressure) can

also be estimated using results from DMT, vane or CPTU in situ tests.

The compression index C_c of Recife clays are related linearly to e_0 and the research sites are highly

compressible deposits with great differences between the layers of the deposits, presenting average values

for C_c of 1.55 and 0.75 (RRS1) and 2.4 and 1.2 (RRS2), respectively.

The geotechnical compressibility and strength parameters of Recife clays are heavily affected by

sampling disturbance. The ratio or relationship between some parameters and sample quality is presented. Coefficients of consolidation in vertical and horizontal directions have been obtained by laboratory

and in situ tests (CPTU and DMT) and the ratio c_h / c_v obtained with oedometer tests was between 1 to 2.5

with $c_h \sim 2.0\text{--}2.5 \times 10^{-8} \text{ m}^2/\text{s}$ in the normally consolidated range. Piezocone values of c_h were at least

twice higher than laboratory c_h values. DMT c_h values were higher than the c_v laboratory oedometer

results ($c_h / c_v = 1$ to 3).

The Recife clays undrained strength was obtained using laboratory triaxial UUC and CIUC ($\sigma'_c \sim \sigma'_{oct}$)

tests and in situ vane tests and the results are in general in the range of 10 to 55 kPa. Values of S_u can

also be well estimated using DMT, Piezocone and PMT tests.

An embankment failure analysis confirmed the need to apply the Bjerrum's correction factor to the

in situ vane test results to obtain S_u values for use in design. Measured effective strength parameters

of Recife clays in the normally consolidated range were: RRS1 - c' (kPa) = 0 and $\phi'(\sigma) = 25$ -28 and

RRS2 - c' (kPa) = 0 and $\phi'(\sigma) = 25$ -27. As for most natural clays, the limit state curve of Recife clay

is not centered on the isotropic axis and it is heavily affected by sampling disturbance. The Young's

modulus measured at 50% of the maximum deviator stress and the ratio E_u / S_u showed dispersion and

was practically "constant" with depth ($E_u / S_u \sim = 270$).

The creep behavior of Recife clay was observed in oedometer laboratory tests showing important

deformation due to secondary compression.

The piezocone and DMT were shown to be a useful tool for providing a "continuous" estimated profile

of a number of geotechnical parameters, such as K_0 , OCR, M , undrained strength, and c_h . The Ménard

pressuremeter (PMT) was also carried out and provided good results for S_u . A quantitative comparative

study was done showing the uncertainty associated with the prediction of these geotechnical parameters

from in situ tests. In Recife the clay deposits, the in situ tests can be used as a complement of a reference

test investigation or to define approximate values that can be used for preliminary designs.

The practical applications were useful in providing the actual behavior in the field on an embankment,

shallow and piled foundations on soft clay. Embankment stability analysis, settlement of the shallow

foundation, lateral and vertical load capacity of the piled foundation, including a buckling study of the

steel pile caused by lateral displacement in soft soil, were studied using the geotechnical characterization

from laboratory and in situ tests. The results confirmed the good potential to predict the field behavior

in the Recife soft clays.

ACKNOWLEDGEMENTS

A special acknowledgement is due to the National Research Council (CNPq) and CAPES for the fellow

ships granted to researches and graduate students, to CNPq through the PRONEX program, instrumental

to obtain field and laboratory data. Special thanks for all research members of the Geotechnical Group -

GESEP/DEC/UFPE that participated in the research projects. Thanks are also due to Ph.D. students

Maria Isabela Marques da Cunha Vieira Bello and Karina Cordeiro de Arruda Dourado, and M.Sc

student Juliana Albuquerque Biano de Lemos, who helped in the final draft and revisions.

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Characterisation and Engineering Properties of Natural Soils - Tan, Phoon, Hight & Leroueil (eds) © 2007 Taylor & Francis Group, London, ISBN 978-0-415-42691-6

Characterisation and engineering properties of Dutch peats

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ABSTRACT: The surface layers in the lowlying parts of the Netherlands which form the bread and

butter of Dutch geotechnical engineering, consist for the greater part of peat and soft, usually organic

clays and silts. This paper draws on data from various locations in the western Netherlands to synthesize

a picture of the behaviour of the peats, but crosses the borderline into the organic soils here and there.

The formation, composition, structure, state, index and engineering properties of the soils are described.

They are characterised by low stiffness, high compressibility and large creep, and high effective strength

and normalised undrained strength parameters. Bulk density is low and has a negative correlation with

the foregoing characteristics. Present design methods inadequately deal with the characteristic traits of

these soils and this is briefly touched upon. Laboratory test procedures for index testing are dealt with in

some detail. Typical responses in oedometric and triaxial tests are presented and correlated with index

properties. The compression and disturbance of peat during sampling is considered, and in-situ testing

is briefly discussed. Some engineering problems specific to these soils are discussed and a case history

of a failed canal dyke is presented.

1 LOCATION AND DISTRIBUTION OF DUTCH PEATS AND ORGANIC SOILS, AND THEIR LOCAL AND GLOBAL IMPORTANCE

The lowlying areas of the Netherlands occur in the west and the north of the country and are underlain

by Pleistocene sands which dip in north-westerly direction to depths in excess of 20 m along the present

coastline. The map in Figure 1 shows the distribution of the Nieuwkoop Formation which contains most

of the peats and soft organic clays in the Netherlands. Figure 2 is an west to east cross section through

the country which illustrates vertical distribution and thicknesses of the various Holocene layers. At the

base is a compressed layer of peat which occurs quite generally throughout the area. It is from 10-50 cm

thick. Above this layer marine clay was deposited, and peat was formed. The clay is often organic; the

peat is generally uncompressed or covered by a thin layer of clay. Further inland, the clays are fluvial

deposits formed in back swamps behind natural river levees.

The initial occupation of the Dutch lowlands took to higher grounds rising above the swampy terrain

such as natural levees along waterways, coastal barriers and isolated hills and dunes. From there swamps

were drained. The population explosion in the industrial era has resulted in widespread occupancy of

surrounding terrain. The very poor foundation conditions are combatted by backfilling with sand, and this

technique is used on a large scale for harbours, industrial sites and urban development. Subsidence due

to drainage and backfilling amounts to approximately 1

cm/year in many areas, and presents increasingly

problematic drainage conditions.

The softest and most organic subsoil coincides with the most densely populated areas of the Nether

lands, and therefore the properties of these soils are of paramount importance to geotechnical engineering.

Many older buildings have suffered severe tilting due to misjudgement of the foundation conditions, and

piling was developed early on to by-pass the soft layers altogether. The piles reach into the underlying

Pleistocene sand and are the preferred solution for foundations of buildings, bridges, and recently, high

speed railway lines. The soft soils remain to be dealt with in dykes and waterways, road and railway

embankments, excavations and tunnels.

The peat and the organic clays are juxtaposed throughout the region and must usually both be dealt

with at the same time. There is a smooth transition not only in composition but also in geotechnical

behaviour from the one to the other. Important studies on peat have been performed by Hobbs (1986,

Figure 1. Distribution of Nieuwkoop Formation, and locations of referenced sites. A =Amsterdam, R = Rotterdam,

H = Hague, M = Marken, OVP = Oostvaardersplassen, RW = Rijkswatering, W =Wilnis, Z = Polder Zegveld,

S = Sliedrecht.

Figure 2. Cross section (west to east) through soft soils in the central part of the country. From Ebbing et al. (2003),

after Berendsen (1996) and Cleveringa (2000).

1987) and Landva et al. (1980, 1983a, b, c, 2006 in the present volume), and overviews of peat behaviour

are given in the Muskeg Engineering Handbook (McFarlane 1969), Carlsten (1988), the Peat Engineering

Handbook (Noto 1991) and by Den Haan (1995, 1997). Studies of organic clay behaviour include those

on Juturnaíba clay by Coutinho & Lacerda (1989), north German and Dutch soils from Schwerin, Berlin

and Rotterdam by Krieg (2000), Queenborough clay from the lower Thames estuary by Jardine et al.

(2003), and Recife clays (Coutinho 2006, in the present volume).

The common traits of behaviour of these soils are low density, high effective stress strength parameters,

high normalised undrained strength, strong creep susceptibility etc. and the Dutch peats and organic clays

will be shown to fit into this pattern. The organic content has a strong influence on these properties and as

it usually varies strongly with depth, it is useful to consider the role of the organic content. The borderline

between peats and organic clay is not easy to draw, and consequently in this paper, some data on what

may be classified as organic clay also appears.

2 GENESIS

The geological development of the Netherlands during the Holocene period was dominated by the rise

of the sea level, which is conjectured to amount to approximately 120 m in the last 10000 geological

years, the last 20 m of which occurred in 7800 years. Soft soils and peat accumulated in a lagoonal tidal

basin setting, protected from the North Sea by a coastal barrier. The accumulation kept pace with the

Holocene sea level rise, resulting in a wedge of deposits extending 30-50 km inland and which is locally

well over 20 m thick at the present coast. This wedge is

crossed locally by various forms of fluviatile,

estuarine and tidal inlet deposits composed of mostly sand and clays. Groundwater levels were forced upwards along the encroaching sea and a marshy coastal wetland

migrated landwards by the rising sea levels. Various types of peat developed, depending on ground

water level, salinity gradients and nutrient supply. Further variations were developed by differences in

compaction due to the overburden, weathering and other ageing processes. The peat which accumulated

directly on the Pleistocene substratum of sand or loam, at the base of the Holocene sequence, is more

compact, altered, frequently more weathered, and usually contains significant amounts of mineral matter.

It is referred to as basal peat. Extremes in terms of nutrients are wood peat and sphagnum (moss) peat. In

the nutrient rich riverine areas peat with much very coarse wood accumulated, usually with a significant

mineral matter content. In nutrient poor environments, sheltered and away from open water and seepage

sources (fringing upland terrain), sphagnum moss peat accumulated. Peat accumulated at the landward margin of the lagoon, and also throughout most of the area during

periods in which the barrier system was largely closed. Clay, organic clay and sands accumulated during

several periods when the coastal barrier was unstable and was breached by numerous tidal inlets and

enlarged estuaries. The peat sequence is interrupted by these marine and related clastic deposits during

those periods. In the riverine area patches of peat accumulated in abandoned channels and backswamps.

Peat accumulation continued up until Roman times, with a height of almost 2 m above sea level being

reached. Drainage of the swamps ended peat growth, and was

undertaken in earnest from around the year

1000AD, resulting in progressive areal subsidence and the necessity to develop protective embankments.

This lowering of the terrain with the continuing sea level rise has resulted in increased flooding and the

deposition of clays and sand on top of the peat in flood prone areas near rivers and tidal inlets. Eventually,

in the course of a few centuries, the lowest peat terrains had to be abandoned, and quite some of these

were mined later as a source of turf fuel. Peat accumulation and mire formation is described in the literature by the wetland succession. It is

the slow process of growth, dying off and accumulation of plant material in a succession through various

morphological stages and associated plant communities. The ideal succession is autonomous and goes

from lake filling in a nutrient-rich environment by e.g. reeds and sedges (fen peat), to a transitional stage

involving e.g. wood peat, and finally the raised-bog stage involving heathers and mosses (bog peat). This

has occurred only very locally (patches in the order of 10 m²) in the Netherlands, and the overriding

influence on peat formation has been the rise of the sea level, the associated rise of the groundwater

level, and the integrity of the coastal barrier.

The present-day IJsselmeer lake and the neighbouring large polders of Flevoland together formed the

Zuiderzee tidal basin up to 1932, when it was reclaimed. The Zuiderzee area experienced several stages

of transgression, regression, peat growth and erosion. Around the time of the arrival of the Romans,

smaller lakes merged into lake Flevo, which gradually expanded further. Eroded peat was deposited on

the lake bottom, followed by fine silt in deltas of rivers entering the area. These Almere deposits therefore

are characterised locally by an upwards diminishing content of organic matter. The connection to the

sea through the Waddenzee tidal flats to the north widened further around 1600 AD and the Zuiderzee

deposits were formed in brackish to salty water.

The continued rise of the sea level and ground water level and the subsidence following cultivation of

the swamp areas has submerged much of the peat in the western Netherlands, and formerly raised bog

peat areas now lie under groundwater level. Fen peat is however the main constituent. Fen peats often

have a relatively high mineral content. They tend to be more humified, and have lower water contents

D	dec	w f	[k g / k g]
5	6	7	8
9	10	11	12
13	14	15	10
20	30	40	50
60			

Figure 3. Dependence of total water content w f on the degree of peat decomposition D dec : (1) highbog peat;

(2) transient peat; (3) fen peat. Reproduced from Amaryan (1993).

Figure 4. Historical subsidence of peat areas.

than bog peats, which is well illustrated by Amaryan (1993), see Figure 3. Fibrosity of peat decreases as

humification increases. The historical rate of subsidence in the drained peat areas has been shown to amount to less than

2 mm/year over 1000 years, reducing the freeboard above mean sea level from 2 m to zero, Figure 4.

Subsidence rates drastically increased after the introduction of forced drainage to accommodate agricul

tural interests. The subsidence consists of components due to weathering of organic material, shrinkage

and consolidation. Weathering is responsible for some 70%

of the total, in present conditions: increasing

of course as subsidence progresses and all peat is eventually lost. Conservation of peatland has become

a goal which is contested in the political arena, and subsidence rates are being brought under control in

many polders as a result. Nevertheless, many peat areas now lie at levels some 2 m below sea level, and

reclaimed former peat lakes can lie as low as 6 m below sea level. Ground water in the lowlying polders is discharged into surrounding canals and eventually into the sea.

Canals often stand out from the surrounding polders, either by subsidence of the polder or reclamation

of former lakes. The dykes which retain the canals quite often consist of the original ground which in

many cases is peat. An estimated 4000 km of such peat dykes exist.

3 COMPOSITION AND CLASSIFICATION

3.1 Botanical composition of Dutch peat

Sedentary peat (formed in place) consists of reed and sedge peat, wood peat and moss peat. The eroded

and deposited organic material in which plant remains can still be distinguished, is given names such as

“meermolm” (organic lake mud), “detritus” and “verslagen veen” (detrital peat). “Rottingsslijk” signifies

sapropelium in which the organic material is strongly decomposed. Detrital fragments, sapropelium and

sedentary remains occur as constituents of organic clay together with mineral constituents.

3.2 Classification of peats and organic soils

Many different classification systems for peat and organic soil exist in the literature, and not one is

generally accepted. Andrejko et al. (1983) and Landva et al. (1983a) together list not less than 12

different systems used for the classification of peats and organic soils, based solely on organic content.

Their common aspect is the definition of peats at the high end of the organic content scale, organic soils

at the low end, and so-called muck in between. The boundaries between these soils are put at widely

varying organic contents however. Most of these systems serve soil-scientific, calorific and geologic

purposes and have no special significance for civil engineering. It is clear that organic content alone is not

sufficient to fully characterise a peat for geotechnical purposes. Other potentially useful classification

parameters are: botanical composition, degree of humification, water content, bulk density either dry or

wet, specific mass of the compounded organic and inorganic constituents, fibre content and fibre-size

distribution, (anisotropic) shrinkage, tensile strength, and if at all possible to determine, the Atterberg

plasticity limits. Chemical parameters such as pH and contents of organic carbon and carbonates are of

interest for special purposes, especially when contemplating soil stabilisation techniques such as deep

mixing with cements etc. The composition of the organic matter in terms of bitumen, hemicellulose,

cellulose, lignin, humic and fulvic acids etc may also be of interest.

The von Post classification system, of which Hobbs (1986) provides an English-language summary,

is best known for its definition of degree of humification, going from H 1 for intact, young peat to H 10

for completely decomposed peat. The method is based on simple manual methods suitable for use in the

field. The von Post system also includes classification

according to botanical composition, water content,

content of fine and coarse fibres, and content of woody remnants. Landva & Pheeney (1980) modified

Von Post's system, and Hobbs extended it with categories for organic content, tensile strength, odour,

plasticity and acidity. The Dutch guideline for the geotechnical classification of peat (1996) adopts a

similar system. Basically the von Post classification system is adopted, extending it with organic content

and designation of the kind of inorganic material encountered. The ten humification degrees are brought

back to only four. In Figure 9, five degrees are used, and in Figure 3, Amaryan's degree of decomposition

D dec appears to be continuous. Unluckily, D dec was not defined. In the systems adopted by von Post, Landva and Pheeney, Hobbs and the Dutch guideline, each

characteristic is designated by a letter, and the degree to which a characteristic is present, is designated

by an index. Thus, with Hobbs,

designates Sphagnum (S) moss peat with moderate humification degree (H), water content (B) less than

500%, a high content of fine fibres (F), a low content of coarse fibres (R), no wood (W), a moderately high

organic content (N), a low tensile strength in the vertical direction (TV) and a moderate tensile strength

in the horizontal direction (TH), no smell (A), a detectable plastic limit (P) and a neutral acidity (pH). Such elaborate formulae, while potentially useful for the comparison of different soils and for correl

ation purposes, require much training and effort to construct and apply. In the Netherlands at least, they

have not yet been applied. More simple descriptions could fall back on the essential data: bulk density,

water content, main botanical type and perhaps some

indication of the degree of humification. In the

authors experience, organic soil is well characterised by bulk density. High water content peat the bulk

density of which cannot be distinguished from the density of water, is possibly better characterised by

water content, but insufficient data exists to develop correlations for these peats.

3.3 Nomenclature of Dutch organic soils

The nomenclature of Dutch organic soil is based on 2 ternary diagrams in which zones discern between a

main constituent (either peat, clay, silt or sand) and secondary constituents in varying degrees, Figure 5.

Capital letters designate the main constituent, and lower case letters give the secondary constituent

followed by a number to indicate its relative importance. E.g. Vk3 indicates very clayey peat, and Ks2h2

indicates moderately silty, moderately organic clay. The slanted lines in the first ternary diagram reflect a

dominance of the clay fraction in the properties of the mixture. The borderline between peat and organic

soil lies at 15-25% organic matter.

Figure 5. Ternary classification diagrams for soft soils. (Reproduced from NEN-5104:1989 with permission of

NEN, Delft.) Table 1. Indicative bulk density of Dutch organic soils. Bulk density ρ (t/m³) Designation
Description nr Mean Range V_m Peat, poor in mineral matter
98 1.06 0.95-1.13 Vk1 Slightly clayey peat 21 1.11
1.02-1.17 Vk3 Very clayey peat 16 1.18 1.09-1.25 Kh3 Highly
organic clay 13 1.27 1.20-1.35 Kh2 Mod. organic clay 30
1.37 1.26-1.51 Kh1 Slightly organic clay 60 1.45 1.30-1.60
Ks Slightly to highly silty clay 89 1.51 1.40-1.60 Many
properties of Dutch organic soil are found to correlate
with total bulk density ρ as determined

from borehole samples. Table 1 was constructed from 350 soft samples (defined as having a bulk density

lower than 1.60 t/m^3) taken from the upper 10.5 m along the route of the new High Speed Railway near

Rijpwetering between the Hague and Schiphol, and provides a link between nomenclature and bulk

density. Standard deviations of the bulk density are in the order of 5%. Here we already see the gradual transition from peats to organic clays. The bulk density of peat as

measured in the laboratory can fall short of 1.0 when it loses water due to a coarse fibrous structure, or

when it contains air and gasses. Peats with a bulk density near 1.0 have a high content of organic material

and strongly varying water contents, and correlations of their properties with bulk density are evidently

not useful.

3.4 Plasticity chart

The plasticity limits are not in general use in the Netherlands as a means of characterising soft soils. In

peats they are not determined due to the presence of fibres. Figure 6 nevertheless brings together data

acquired over the years from a number of sites and soils for organic clays. The high values of w_L will be noted. Most classification systems are valid only for $w_L < 100\%$.

The highest values have the highest organic contents, as is confirmed in Figure 7a. The position in the

Casagrande chart is generally above the A-line for all soils irrespective of organic content. 0 50 100 150 200 250 300 350 0 100 200 300 400 500 w_L [%] I_p [%] A-line
Betuwe railway OVP OVP detritus clay Rotterdam (Krieg 2000)
Bergambacht Queenborough natural Queenborough air dried
Skempton & Petley, peats

Figure 6. Plasticity chart of various organic soils. 0 100 200 300 0 10 20 30 N [%] w_L [%] Betuwe Railway Skempton & Petley, clays $w_L = 30.5 + 8.0N$ 0 100 200 300 400 500 0 100 200 300 400 w [%] w_L [%] Betuwe railway HSL railway Skempton & Petley, peats

(a) (b)

Figure 7. Dependency of liquid limit on loss-on-ignition N (a) and on natural water content w_0 (b).

The majority of the Dutch organic clays are slightly to moderately silty. E.g. in Table 1 above, 64 of

the 192 clay samples are slightly silty, and 117 are moderately silty. However, even very silty material

plots in the same region above the A-line as evidenced by the organic clay from Rotterdam studied by

Krieg (2000). This was composed of 73% silt, 8% fine sand, 19% clay and 16% organics, which classes

the material as Lz_{1h3} , that is slightly sandy highly organic silt.

The organic clay from Queenborough studied by Jardine et al. (2003) plots in the same region as the

Dutch organic soils. Clay content was 58%; silt content 42%. Air drying reduces w_L and the Casagrande

point translates downwards parallel to the A-line.

The marked tendency for all samples to plot above the A-line contradicts most classification systems

where organic soils are zoned below the A-line. Skempton & Petley (1970) show a Casagrande chart

for a variety of British peats and peat/clay mixtures. The peats plot within the band shown in Figure 6,

higher w_L correlating with higher organic content. The organic clays plot on the A-line and the lower

boundary of the shown band. The low I_p values were put down to the organic matter contributing greatly

to water-holding capacity, but much less so to plasticity. Their results are difficult to reconcile with the

present results, but may possibly be explained by the experimental difficulty of determining the plasticity w_p .
0.0 0.2 0.4 0.6 0.8 1.0 1.2 1.4 1.6 1.8 2.0 $p [-]$ R e l a t
i v e v o l u m e o f p h a s e s Unfilled markers: Betuwe
railway Full markers: Polder Zegveld

Figure 8. Phase diagram of peats from Polder Zegveld and organic soils from Betuwe railway project. Phases from

bottom upwards: anorganic - organic - water - gas.

limits (especially the plastic limit) of highly organic soil and peat. Figure 7a shows reasonable agreement

of the $w_L - N$ correlations between Skempton & Petley and the present results. w_L correlates reasonably

well with w_0 , Figure 7b, and Skempton & Petley's peats are more or less an extension of the relation for

organic clays. Only occasionally is w_0 above w_L . Some organic clays do plot below the A-line. Two examples are shown: a highly organic detritus

clay layer from Oostvaardersplassen between 3.0 and 3.5 m depth, and a similarly highly organic clay

from Bergambacht (riverine district) between 8.5 and 9.5 m depth. The large range of values in the

chart for these clays is a result of the strongly varying organic contents: from 10% to 40% for the

Oostvaardersplassen layer and from 20% to 30% for the Bergambacht layer. Bulk density is between 1.1

and 1.17 in both cases. These soils border on being clayey peats, and in fact Table 1 indicates they are. The liquid limit of the Betuwe tests in Figure 6 were determined with the Swedish fall cone; the

remaining tests with the Casagrande device. There does not appear to be an influence of test type on the

location in the chart.

3.5 Phase diagram

The relative amounts of mineral matter and organic matter in Dutch organic soils can be read from

Figure 8. It gives the relative volumes of the anorganic, organic, water and gas phases as a function

of natural bulk density ρ , separately for Polder Zegveld

and the Betuwe railway project. These were

determined from measurement of bulk density, water content, loss on ignition and specific density of

solids, each on one and the same sample. As bulk density increases, the anorganic phase increases and

the organic phase diminishes; both approximately in a linear fashion.

3.6 Polder Zegveld

A typical peat profile of the western peat district was studied by El-Amir (1989). It was obtained from

continuous 66 mm Begemann borings in Polder Zegveld near Woerden and is shown in Figure 9. The

soils classification and determination of the degree of humification were performed by P. Krajicek. Above the Pleistocene sand base, there is a 6.5 m thick sequence of completely decomposed peat,

sedge, reed, sedge, and wood peat, overlain by a thin layer of completely decomposed peat. This is

covered by a surface clay layer of 1 m thick. Thin detritus and sedge layers occur within the wood peat.

Reed and sedge peat usually occur together. No sphagnum peat was encountered here. The degree of humification was evaluated on a scale of 1-5. It varies throughout the depth, but

there is a faint correlation with botanical peat type. Three horizons of upwards increasing humifica

tion appear to be associated with sedge content. The wood peat tends to be highly humified; the reed

peat is lightly humified. The water content varies from approximately 350% to 850% and shows a 0 2 4 6 8 10 w 0
[-] 0 1 2 3 4 5 6 7 8 0 1 2 3 4 5 Degree of humification D
e p t h b e l o w N . A . P . [m] 0.9 1.0 1.1 1.2 p [-]
0.0 0.4 0.8 N [-] Reed peat Sedge peat Wood peat Completely
decomposed peat Muck Sand base

Figure 9. Borehole description and depth profiles of various index parameters, Polder Zegveld.

Figure 10. Structure of decomposed wood peat, N = 67% (a) and reed peat, N = 83% (b), Polder Zegveld.

weak negative correlation with the degree of humification. The profiles of water content and loss-on

ignition show a strong correlation: mutually, and also with organic content. The reed peat was coarse

fibrous and its low bulk density ($<1 \text{ t/m}^3$) can therefore be explained by loss of water from large

voids. Figure 10 gives photographic cross sections of borehole samples of a decomposed wood peat from

4.77 m - N.A.P. with an organic content of 67%, and a reed (sedge) peat from 6.20 m - N.A.P. with an organic content of 83%.
 Wood peat Reed peat Sedge peat
 -0.04 0.00 0.04 0.08 0.12 0.16 0.0 0.2 0.4
 0.6 0.8 1.0 $\epsilon_{\text{vol}} [-]$ $\epsilon_v [-]$ $\epsilon_h [-]$ Clayey wood peat
 Decomposed wood peat

Figure 11. Anisotropy of peat revealed by shrinkage tests, Polder Zegveld.

organic content of 83%. The highly heterogeneous and anisotropic nature of the latter is immediately

apparent. Reed and sedge fibres of varying sizes but with a predominantly horizontal orientation are

mixed into a matrix of fine fibres and mineral matter. The decomposed wood peat has coarse elements

such as twigs and roots mixed into it, but the matrix is homogeneous. An indication of the degree of anisotropy of the structure of peat is quickly obtained from the shrinkage

of a sample in the axial and radial directions as it dries to the air (Hobbs 1986). Figure 11 shows the

anisotropy found for the different botanical peat types of Polder Zegveld. There is a clear hierarchy

in anisotropy, in the order reed peat - sedge peat - wood peat. Decomposed wood peat dries without

distortion, indicating that decomposition reduces structural anisotropy.

4 STATE AND INDEX PROPERTIES

Correlations between various state properties and index properties are given in this section based on data

from various sites. The state properties considered are bulk density ρ , natural water content w_0 , natural

voids ratio e_0 ; index properties are the solids density ρ_s and organic content N . Some comments on the

determination of these properties are given where appropriate.

4.1 Loss on Ignition and Organic Content

During a Loss on Ignition test, a sample which has been previously dried, is combusted at high tem

peratures, and the loss of mass or Loss on Ignition value N is determined. It correlates closely with the

organic content OC . Various ashing temperatures have been used in the Netherlands in the past: 500,

550, 800 and 950 °C. Corrections are necessary to account for the evaporation and decomposition of

non-organic constituents such as bound water in the clay, and CO_2 from calcium carbonates. Oxidation

of FeS_2 in poorly aerated soil can result in a mass increase. The former Soil Survey Institute (Stiboka: Stichting Bodemkartering) used 550 °C and the following

correction formula:

where N is Loss on Ignition, and L , S and Z are the mass proportions of material $<2 \mu m$ ("lutum"), silt,

and sand respectively. Skempton and Petley arrived at another correction formula:

where proportions are relative to unity. The corrections $OC - N$ in both formulae are negligible for high

values of N , and differ by only a few percent on the borderline between peats and organic soils in the

ternary diagram above. The ease of determination of N makes it the property of choice for correlation

and characterisation above direct determination of OC by chemical methods. The present guideline used

in civil engineering prescribes 500 ° C during 4 h. 0.0 0.5 1.0 0 20 40 60 80 100 Loss-on-Ignition N [%] m t [%] m t = (m s ,85° - m s ,105°)/m s ,105° Polder Zegveld

Figure 12. Loss of dry mass by drying from 85 ° C to 105 ° C relative to dry mass at 105 ° C, Polder Zegveld. 0 200 400 600 800 1000 0.0 0.5 1.0 1.5 2.0 p d , p[-] w 0 [%] HSL Rijkswatering 's-Gravendeel Betuwe railway Polder Zegveld Bergambacht

Figure 13. Correlation of wet and dry bulk density with natural water content for various Dutch peats and organic soils.

4.2 Water content

There is a general fear that the standard drying of soil at 105 ° C during 24 hours will lead to charring of

the organic components in peat, thus producing too large figures for water content. Lower temperatures,

between 50 and 95 ° C are therefore advocated by some. Skempton and Petley investigated these effects,

and concluded that the loss of organic matter at 105 ° C is insignificant, while drying at lower temperatures

retains small amounts of free water. This was corroborated by El-Amir in tests on Polder Zegveld peats,

Figure 12. m t is the ratio of the apparent loss of dry mass on increasing the drying temperature from

85 ° C to 105 ° C, to the apparent dry mass at 105 ° C. It appears to peak at N ≈ 70%, which was also found

by Skempton and Petley. m t increases if lower values than 85 ° C are taken, but remains below 3% even

at 65 ° . Such small values justify the choice of 105 ° C as drying temperature.

4.3 Water content vs. bulk density

Figure 13 plots natural water content against natural bulk density, both wet (ρ) and dry (ρ_d). The tight fit

of $w_0 - \rho_d$ is remarkable considering the number of data points and sites involved. Very similar relations

are given in the literature, e.g. by Farkas & Kovács (1988), Kabai & Farkas (1988) for Hungarian peats,

and Marachi et al. (1982) for Californian peats. It is given by The $w_0 - \rho$ relationship is also relatively tight, albeit much less so than for $w_0 - \rho_d$. The relationship

between $w_0 - \rho$ is given by

where ρ_s is solids density and S is degree of saturation. The scatter in the figure is due to variations in

ρ_s and S , lower ρ_s and lower S bringing (w, ρ) points downwards and to the left.

4.4 Specific gravity

Specific gravity in organic soils is affected by the organic constituents, and cannot simply be set to

somewhere near 2.65-2.75 as in mineral soils. Cellulose has a specific gravity of approximately 1.58,

while for lignin it is approximately 1.40. These low values reduce the compounded specific gravity of

organic soils. The variability of organic content and organic constituents in any peat deposit requires its

separate determination whenever voids ratio is to be calculated. Use of automatic helium gas displacement

pycnometers allows cheap and fast determination of specific gravity. Kerosene has been used in the past

in fluid displacement methods, but for peats, more satisfactory results are obtained with hexane. Skempton & Petley produced a very sharp correlation between specific gravity of a suite of peats and

organic soils and their ignition loss, and fitted the relation by

where p_s is compounded specific gravity and N is ignition loss. The numbers of 1.4 and 2.7 correspond to

the separate specific gravities of the organic and inorganic fractions respectively. Ignoring the correction

between ignition loss and organic content which has been applied in equation (6), Figure 14 gives an

immediate relationship between N and p_s for soils from a variety of sites. The close correlation of Skempton & Petley is retained. There appears to be slightly more scatter

as the mineral content increases, possibly due to increasing divergence between organic content and

loss-on-ignition. In establishing the correlations of Figure 14, both characteristics have been measured on the same

piece of material. Peat deposits are so extremely variable that quantities measured on material only

centimeters apart, need not have any correlation. In this particular case, it is quite easy to homogenise a 1.2 1.6 2.0 2.4 2.8 0.0 0.2 0.4 0.6 0.8 1.0 Loss-on-Ignition N [-] p_s [] Betuwe railway Polder zegveld Bergambacht Oostvaardersplassen Rotterdam (krieg 2000) Slidrecht 2.746 1 - N 1.354 1 N $\pm$ p_s

Figure 14. Correlation of solids density p_s with Loss-on-Ignition value N .

dried sample by pestle and mortar, and take one subsample for specific gravity testing, and another for

ignition loss. If this sample is first cut to fit an oedometer ring, bulk densities and water content are also

established on the same material, extending the possibilities for determining reliable correlations.

4.5 Water content vs. voids ratio

Natural water content and voids ratio exhibit a reasonable correlation as seen in Figure 15. The linear

onset of the correlation near the origin corresponds to a

solids density of approximately 2.75. At higher

water content, the lower solids density results in reduced voids ratio.

4.6 Variability of peat deposits

Both laterally and vertically, peats show large variability. Borings taken by Sikder (1994) at 2 m centres

in a peat area in Delft yielded strong variations in loss-on-ignition N as shown in Figure 16. The vertical

scale in the figure is ten times larger than the horizontal scale. As the areas of equal ignition loss

in the figure are approximately circular, it may be concluded that the vertical variability is ten times

more intense than the lateral variability. This extreme variability on the decimeter (vertical) resp. meter

(horizontal) scale probably also exists at larger and smaller scales as well. Predictions of peat behaviour

should account for this variability. 0 4 8 12 16 0 2 4 6 8
10 w 0 [-] e 0 [] Polder Zegveld Betuwe railway
Sliedrecht

Figure 15. Correlation of natural water content and voids ratio.

Figure 16. Variability of ignition loss value in an organic soil deposit, Delft.

5 STRUCTURE

The role and dispersion of mineral matter in peat has been well analysed by Hobbs (1987). There it is

shown that even at an organic content as low as 27.5%, any mineral matter can easily be accommodated

within the voids of the peat. This includes the bulked volume of the mineral matter, be it silt with a low

bulking factor or clay at its liquid limit. Thus where peat is sedentary (formed in place), the washed-in

mineral matter is absorbed into the vegetable structure,

and behaviour is dictated by the latter. The Figure

of 27.5% organic content corresponds approximately to a natural water content of 250% and Hobbs shows

that geotechnical behaviour as expressed in the coefficient of secondary compression $C_{\alpha} = \frac{de}{d \log(t)}$,

shows a clear transition around this value. Note that the Dutch distinction between peat and organic clay

at an organic content of 15-25%, is at the clayey side of Hobbs' findings.

6 ENGINEERING PROPERTIES

6.1 1D compression

Settlement analysis remains an important core activity of geotechnical engineering in the Netherlands

due to the soft nature of the subsoil. Due to the large compressibility and creep ability, the bi-linear stress

strain model is too simple. A first adaptation is to make use of natural (Hencky or logarithmic) strain to

cope with the concavity of the stress - strain curve which invariably develops at large strains. Figure 17a

illustrates this for the 24 h compression curve of peat sample 25D from Polder Zegveld (4.15 m - N.A.P.).

Natural strain is given by

and equals $-\ln(1 - \epsilon_{lin})$. It retains the notion that stiffness is proportional to effective stress:

where b is the slope of the virgin compression curve. 10
 100 1000 10000 vertical stress σ_v [kPa] n a t u r a l s
 t r a i n ϵ_H [] 1e-4 1/s 1e-4.5 1/s 1e-5 1/s 1e-5.5 1/s
 1e-6 1/s 1e-6.5 1/s 1e-7 1/s ** creep isotaches * total
 strain isotaches distorted isotaches slope $b = \frac{de_H}{d \ln \sigma'_v}$
 * ** 0.0 0.2 0.4 0.6 0.8 1.0 1.2 0.0 0.2 0.4 0.6 0.8
 1.0 1.2 10 100 1000 10000 vertical stress σ_v [kPa] N a t
 u r a l s t r a i n ϵ_H , l i n e a r s t r a i n ϵ_{lin} [
] linear strain natural strain Polder Zegveld sample 25D
 slope $b = \frac{de_H}{d \ln \sigma'_v}$ (a) (b)

Figure 17. Linearisation of virgin 24h oedometer

stress-strain curve by natural strain (a) and isotaches (b) of a

peat sample from Polder Zegveld. Further, creep must be accounted for. Isotache models such as developed by Leroueil et al. (1985)

for inorganic clays are also accurate in organic clays and peats. They are now readily available and are

becoming the norm for settlement analysis, and correlations have been developed for the parameters

involved. Briefly, isotache models define creep rate as depending uniquely on effective vertical stress

$\sigma' - v$ and strain ϵ , and creep isotaches (lines of equal creep rate) are used to visualise this relationship.

These are parallel lines in stress strain space, each lower isotache representing a lower value of creep

strain rate. Figure 17b shows isotaches for sample 25D from Polder Zegveld (Den Haan 1996). The

lower isotaches in the normally consolidated range are taken to represent the virgin viscous behaviour.

The pattern they establish is extended to cover the entire stress-strain plane: higher rates upwards and

lower rates downwards.

The upper isotaches show a different pattern because the abscissa includes pore pressure, and because

the strain rate is increased due to elastic adaptation to the changing effective stress in the primary period.

Elastic strains are formulated by a logarithmic law and added to the viscous strains. Reload isotaches

are distorted due to previous unloading, but this is usually not accounted for.

The slope of the virgin creep isotaches is approximately equal to that of the 24 h compression curve,

b. Although b is an intrinsic soil property, it correlates fairly well with natural bulk density as shown

in Figure 18. The isotache creep parameter which determines the spacing of creep isotaches is given

by $c = \frac{1}{\alpha} \frac{d\epsilon}{dt} H / \ln(d\epsilon H / dt)$. Figure 19a shows a correlation of b/c with natural bulk density for peats and

organic soils from Sliedrecht at a location along the Betuwe freight railway. The values equal the well

known ratio $C_c / C_{\alpha\epsilon}$ and are seen to increase with bulk density. Values as low as 8 are found in peat,

indicating a strong creep ability. The elasticity parameter a with the same definition as b but relating

to elastic strains, is shown as b/a versus bulk density in Figure 19b. This ratio is more or less equal to

C_c / C_r and also generally increases with bulk density. Higher water contents tend to plot in clusters near

$p = 1$. Loss of water before weighing can occur, affecting p and w_0 . Correlations with $p_d = p / (1 + w_0)$,

Figure 20a, could rectify this, but variability is such that w_0 of cuttings can differ appreciably from

the oedometer sample. In the correlation with w_0 in Figure 20b, for both b and c , $w_0 = 250\%$ marks a

boundary above which in particular c hardly increases for higher water contents. This agrees very well

with Hobbs' observations which were referred to earlier.

The remaining parameter of isotache models concerns the reference creep isotache and its position in

stress strain space. It is defined by the rate of $\epsilon'_{ref} = c$ [1/day] and in practice it virtually coincides with

the 24 h virgin curve. Its position is commonly defined by means of the yield stress or OCR, and the

relationship with the initial creep rate of strain ϵ'_0 follows from 0 0.1 0.2 0.3 0.4 0.5 0.9 1.1 1.3 1.5 1.7 p [t/m³] b Polder Zegveld Haastrecht Rotterdam N.Y. org. silty clay (Schmertmann) Soft clay Bangkok $b = 0.326 (\gamma_{nat} / \gamma_w) - 2.11$

Figure 18. Correlation of slope of virgin natural strain isotaches b with bulk density. 0 5 10 15 20 25 30 0.9 1.1 1.3 1.5 1.7 1.9 ρ [t/m³] b/c boring 11.7 boring 16.7 Sliedrecht 0 5 10 15 20 25 0.9 1.1 1.3 1.5 1.7 1.9 ρ [t/m³] b/a boring 11.7 boring 16.7 Sliedrecht (a) (b)

Figure 19. Correlation of b/c (a) and b/a (b) with bulk density. 0 2 4 6 8 w_0 [-] c [-] b [-] c/b 0.00 0.01 0.02 0.03 0.04 0.00 0.01 0.02 0.03 0.04 0.0 0.5 1.0 1.5 ρ_d [t/m³] c [-] 0.0 0.1 0.2 0.3 0.4 0.0 0.1 0.2 0.3 0.4 b [-] c/b Sliedrecht Sliedrecht (a) (b)

Figure 20. Correlation of b and c with bulk dry density (a) and water content (b). σ'_p follows from zero strain on the reference isotache. For correlation purposes a state-independent

parameter in terms of specific volume v is preferred. Writing

where $\sigma'_v1 = 1$ kPa, the specific volume v_1 defines the reference isotache at a stress of 1 kPa. It is akin

to the v_λ of critical state theory. Rather sharp correlations are found with $v_0 (= 1 + e_0)$ and with b ,

see Figure 21. This makes the v_1 , v_0 approach to determining the position of the reference isotache,

attractive, even though it requires the accurate determination of e_0 . The correlations given here fit well

with those given in Den Haan (1992).

0 5

10

15

20

25

30 0.0 0.1 0.2 0.3 0.4 b/v_1 11.7 16.7 v_1 1.336
 $\exp(9.26 b) b$ 0.0622($v_1 - 1.54)$ 0.605 (remoulded clay
 - eq. 15, Den Haan 1992) Sliedrecht 11.7 16.7 Sliedrecht
 y 1.04x 1.37 0 5 10 15 20 25 30 0 5 10 15 v_o/v_1

(a) (b)

Figure 21. Correlation of v_1 with b (a) and v_0 (b).

0.0

0.2

0.4

0.6 0.9 ρ [t/m³] Westrandweg A'dam Sliedrecht Bergambacht
OVP Wilnis Marken

$K_{0,nc}$ 1.1 1.3 1.5

Figure 22. Correlation of $K_{0,nc}$ with bulk density.

6.2 K_0 values

$K_{0,nc}$ values found from a CRS K_0 oedometer are correlated with bulk density for peat and organic soils

from a wide variety of locations in Figure 22. Low values are found in low density material. In the peat,

this can be explained by the resistance of horizontally orientated fibres to lateral extension. The fibres

resist shear distortion of the matrix and experience tensile forces, thereby reducing the lateral stress on

the walls of the oedometer.

A result of a CRS K_0 oedometer test on peat from Sliedrecht is shown in Figure 23a. The constant

rate of strain is interrupted by an unloading/reloading cycle and a relaxation phase. Because σ'_{vh} is also 0.0 0.2 0.4 0.6 0.8 1 10 100 1000 σ'_v [kPa] ϵ_H [-] Betuwe Railway Sliedrecht. ρ 1.29 t/m³ 0.0 0.1 0.2 0.3 1.0 ρ [-] Sliedrecht 1.2 1.4 1.6 ν ↓ ↓ (a) (b)

Figure 23. CRS K_0 test with unload/reload and relaxation loops (a), and correlation of Poisson's ratio ν with bulk

density (b) for Sliedrecht peats and organic clays. 35 45 55 50 60 70 p' [kPa] q [kPa] 72 82 92

102 0 10 15 Relaxation time [h] σ_{vR} σ'_v [kPa] 5

(a) (b)

Figure 24. Relaxation phase of CRS K_0 test yields isotache parameter c (a) and critical state parameter M (b).

measured in these phases, the unloading allows the determination of Poisson's ratio ν . As with K_{nc} the

values tend to be low for the low density material, Figure 23b. In the relaxation phase, the viscosity of the soil expresses itself by reducing σ'_v , and measuring this

allows determination of the creep rate parameter c , e.g. as in Figure 24a. By applying a 2D generalisation

of the isotache model such as the soft soil creep model of Plaxis, the development of σ'_v and σ'_h during

relaxation can be shown to conform to

where $\beta = (1 + \nu)/3(1 - 2\nu)$ and M is the critical state ratio of q/p . This makes it possible to predict

the critical state stress ratio $M = 6 \sin \phi' / (3 - \sin \phi')$ from the unload and relaxation stage in the K_0

oedometer! See Figure 24b. M has been determined for a range of tests from the Slidrecht site, and the results are plotted in

Figure 25 against K_{nc} . As M increases, K_{nc} declines. Shown in the figure is also the well known Jaky

relationship $\sin \phi' = 1 - K_0$ which plots well below the predicted values. The latter are well-fitted by a

line which is also shown as giving the ultimate, fully aged value of K_0 , $K_{0\infty}$. This value follows from the

generalised isotache model by requiring that the elastic component of stress change is reduced to zero

and is given by

where η_{∞} is the ultimate value of the stress ratio q/p' after full ageing. K_0 -CRS tests on organic clays and peat. Bulk density 1-1.5 t/m³ Ultimate, fully aged (2D isotache model) Jaky ϕ' 0.0 0.1 0.2 0.3 0.4 0.5 0.6 1.5 Camclay M [-] K_0 Betuwe railway project 1.7 1.9 2.1 2.3 2.5 nc

Figure 25. K_{nc0} vs. Camclay M value for peats and organic clays from Sliedrecht.

The development of K_{nc0} with time has been the subject of speculation, research and mild controversy

in the 1980's. Schmertmann (1983) posed the question whether K_{nc0} increases or decreases during sec

ondary compression. Although the debate was not conclusive, most evidence pointed to a slight increase.

Figure 25 predicts either depending on the position of the (M , K_{nc0}) state during virgin compression. Den

Haan (2002) showed that this state plots higher if ν is greater. States above the ultimate line would result

in decreasing K_{nc0} during ageing. However, the low values of ν appear to result in a state which is already

on the ultimate line. That is, in these soils, it is predicted that ageing does not increase or decrease K_{nc0}

noticeably.

6.3 Tensile strength

When wedge type failure of a slope in peat occurs, tensile strength of peat is mobilised in the active

zone. Pull-out resistance of horizontally orientated fibres may play a role in this tensile strength. The

Muskeg Engineering Handbook quotes values from Helenelund for tensile strengths of fibrous sphagnum

peat from 3.5-11 kPa, and similar values for sedge peat. These tests were performed dry in a specially

designed tension box. The influence of submergence was investigated by Abebaw (2005). Woody peat

from Krimpen (water content 460-570%) was sampled just below the water table from 0.5 to 0.9 m -

G.L. The sampler was a thin steel ring 40 cm long and 40 cm in diameter and was pushed into the soil

by dead weight and retrieved by excavating around the ring.

In the laboratory, horizontal samples of

66 mm diameter and 100 mm length were obtained by drilling through the wall of the sampler. Tension

caps were fitted at both ends by casting in gypsum. Tension was applied in a standard press, at a rate of

0.2 mm per min. The samples were previously consolidated at approximately 10 kPa. Figure 26 shows

tensile stress vs. strain results for unsubmerged and submerged samples. The curves depart from an

initial compressive state, peak at the tensile strength and drop back to the dead weight of the tension

caps. The initial compressive stress is higher in the submerged samples due to restrained swelling.

The unsubmerged samples have tensile strengths of 3-5 kPa, while in the submerged state the tensile

strength drops to 0-2 kPa. The higher strength in the unsubmerged state is explained by suction pressures

which increase inter-fibre friction and pull-out resistance. In both cases, tensile strength is lower than

the values found by Helenelund. Nevertheless, in back analysis of failures, even a tensile strength of

2 kPa can play a significant role.

6.4 Sample disturbance and heterogeneity

The difficulty of cutting large fibres in peat may cause its compression ahead of the sampler. Helenelund

et al. (1972) used large diameter samplers, 15 to 25 cm in fibrous sphagnum peat, and found that a

saw blade edge in combination with rotation of the sampler reduced the penetration resistance signifi

cantly. Comparison of the stress strain curves of such samples with samples cut by a motor saw was

quite favourable. Samples obtained without rotation of the sampler showed higher strength and lower

Figure 26. Tensile strength tests on peat from Krimpen, in submerged and unsubmerged states.

compressibility than carefully obtained block samples. However, Landva et al. (1983b) found practically

identical shear and consolidation results for apparently undisturbed specimens of fibrous sphagnum

peats from Escuminac, New Brunswick, Canada and specimens prepared from a completely remoulded

slurry. Nevertheless it remains that the disturbance during sampling may reduce water content and thus

give a wrong picture of the soil state in situ. The coarser fibres of reed and wood peat such as are common in the Netherlands increase this risk.

The continuous Delft or Begemann sampler has been the major source of samples for peat investigation

by GeoDelft. This sampler takes continuous samples up to 17 m long by enclosing the sample with a

nylon stocking directly after it has been cut. It is a double tube system with bentonite mud between

the stocking and the p.v.c. liner providing lubrication and lateral support. The sample experiences side

friction only during cutting in the cutting shoe over a length of 58 mm. Sample diameter is 66 mm, the

outside cutting angle is 11° , and the area ratio is 156% which is very high and is due to the width of the

stocking chamber. Generally, the recovery ratio is quite high, with losses of only a few centimeter being the norm for

samples of say 10 m of soft soil. It is difficult to measure the compression of the sample in front of the

sampler, but an indication of it has been obtained by measuring the compression of the "sausage" (the

cut sample enclosed in the stocking) at ground level. This was done for a boring in Polder Zegveld near

Woerden by a level-indicator consisting of a transducer at ground level to which was fitted a length of

tubing containing water. The "sausage" compressed up to 5 cm in the first meter of penetration, but then

stayed more or less at that level during the further boring down to 7.3 m depth. During the fitting of fresh

1 m lengths of sampling tube, the level would tend to rise slightly, but this would be regained quickly

on resumption of penetration. So, although the actual compression in front of the sampler has not been

measured, this result indicates that water content change due to sampling is probably very limited. The

Begemann method has the further advantage that no force is required to take the sample from its liner,

and therefore no change of water content occurs in this stage. The above gives some confidence that peat samples are representative of the in-situ material. Never

theless, the heterogeneity of the soil is such that the sample diameter of 66 mm is unable to capture the

structure of the soil and this must affect the compressibility and especially the shear properties. Organic

clay too is affected by sample heterogeneity and the small size of samples. Triaxial testing is therefore

commonly performed on 66 mm samples without further trimming. Larger diameter sampling and test

ing equipment however is needed to reduce the effects of sample heterogeneity, and ideally, samples are

x-rayed or ct-scanned before the test to be able to discard overly heterogeneous samples. Sample disturbance of organic clay was studied by Den Haan (2003). Samples were taken from 1.6

to 2.4 m depth in the Oostvaardersplassen. Continuous Begemann, thin walled tube and Laval sam

oedometer tests revealed the characteristic structured response (stiff in the overconsolidated branch,

sharp fall after the yield stress, flattening out at larger stress) in the Laval samples but not in the other

samples, but unluckily this sample had a significantly higher density than the others, so that no definite

conclusions could be drawn. It is concluded that the organic Oostvaardersplassen clay is not very sensitive

to sample disturbance.

6.5 Triaxial compression tests

Fibrous peats often allow the effective stress path during shear to reach the tension cut-off, Figure 28.

Landva explained this by pointing out the role of the predominantly horizontal fibres in the sample. These

extend during compressive shear, and impart an internal lateral confining stress to the soil matrix they are

imbedded in, thereby increasing the shear resistance. This phenomenon is frequently met with in Dutch

peats and organic soils. Figure 28 shows 3 CAU tests performed on peat taken at 6.05-6.45 m depth

below a dyke of the island of Marken north of Amsterdam. The material is normally consolidated with

$\sigma'_{p} \approx 45$ kPa and bulk density approx. equal to 1.02. All samples reach the tension cut-off (TCO) where

$\sigma'_{rad} = 0$. Initially the effective stress path moves up the TCO. In this state the samples can drain freely

to the circumference and are therefore no longer undrained. This may explain the increase of strength.

Eventually strength decreases, again along the TCO and at smaller shear strains when overconsolidation

is lower. Shear strains are seen to be large, and this is as it were the price to be paid for the high shear

strength.

A conservative estimate of undrained strength may be taken at the intersection of the initial effective

high ϕ' values in organic clays, decreasing as OC decreased, and Den Haan (1995) shows a similar graph

for the Betuwe railway project. The TCO is sometimes also reached by very organic clay, e.g. as in Figure 31 for Oostvaardersplassen

clay from a depth of 2.4-3.1 m with $\rho = 1.20$ to 1.27 t/m^3 and $N \approx 0.20$ (Tigchelaar 2001). It is a very

organic silty Almere deposit. σ'_{v0} is estimated at 20-23 kPa; $\sigma'_p \approx 45 \text{ kPa}$. The single test which stays

below the TCO had a significantly higher density of $\rho = 1.33$. Unlike the soil which was identified in the

discussion on the position of organic material in the plasticity chart as bordering on being clayey peat,

there is no question about the material in Figure 31 being (very organic) clay. Very similar triaxial test

results were obtained for a very organic Bergambacht clay of the same density range (Tigchelaar 2001),

and also in that case the material is clearly clay. Reaching the TCO is therefore not restricted to peat,

which illustrates the gradual transition of behaviour with bulk density in these peats and organic clays. The undrained strength ratio s_u / σ'_{vc} in triaxial compression and extension is plotted for normally

consolidated samples of Oostvaardersplassen clay in Figure 32 against the bulk density. Only CAU tests

are shown. The ratios are very high in these organic silty clays and the values quickly increase as bulk

density decreases towards the range of peats. Values up to 0.6 are found at low bulk density, while the

denser material yields values of approx. 0.38. This latter is in line with the value mentioned by Mesri

(2001) for triaxial compression of organic clays and silts.
 σ'_1 0 40 80 120 160 σ'_3 0 40 80 120 160 s'_u (σ'_1 ax σ'_3 rad)/2
 [kPa] t (σ'_1 a x σ'_3 r a d) / 2 [k P a]

first stage
 second stage third stage multistage envelopes per test
 regression, first stage $\phi' = 75.0^\circ$ $\rho = 0.96\text{--}1.09 \text{ t/m}^3$ (b)
 (a) (c)

20

30

40

50

60

70

80 0.9 1.1 1.3 1.5 1.7 1.9 2.1 ρ [t/m³] ϕ [°] first stage
second stage third stage 0 10 20 30 40 50 0 20 40 60 Cell
stress first stage, σ'_{c1} [kPa] c' multi stage [kPa]
 ρ 0.96-1.09 t/m³ ↑

Figure 30. Results of multi-stage CIU triaxial tests, HSL railway project near Rijpwetering. (a) ϕ' as function of

bulk density. (b) $s' - t$ end-of-stage values for peat. (c) c' obtained from multi-stage envelopes. p' [kPa] 100 10 0
20 20 30 30 40 50 60 70 q [kPa] TCO

Figure 31. Tension cut-off limiting effective stress path in triaxial compression tests on soft very organic

Oostvaardersplassen clay.

6.6 Multi-stage triaxial tests

An interesting feature of the behaviour of the soft soils is the great difference in ϕ' between the consecutive

stages of the multi-stage test. Figure 30b gives $s' - t$ at the end of the stage for peat. The first stage (not 0.0 0.2 0.4 0.6 1.0 1.1 1.2 1.3 1.4 1.5 ρ [t/m³] s_u / σ'_{vo} compression extension

Figure 32. s_u vs. σ'_{vo} for CAU tests on normally consolidated Oostvaardersplassen clay.

quite yet at failure as the shearing was interrupted at an axial strain of 8% at most) gives a $\phi' \approx 75^\circ$. The

second stage plots below this line, and the third stage lower again, and in Figure 30a the ϕ' per stage

is given. Clearly there is an effect of disturbance in prior stages, accumulating in following stages. The

effect is negligible however in the more mineral soils with higher bulk density. The regression lines of

all stages and densities exhibit very small (negligible) c' values. The use of multi-stage tests in material which is sensitive to disturbance in consecutive stages has a

curious side-effect. It will be noted in Figure 30b that the multi-stage envelopes start on the $\phi' = 75^\circ$ line

and then follow an envelope with a lower angle. These multi-stage envelopes therefore have a c' which

is determined by the choice of the first consolidation stress. This is corroborated in Figure 30c where

σ'_{c1} is the consolidation stress of the first stage. That is, the multi-stage c' is not a material property but

can be manipulated at will. The multi-stage procedure was the norm in the Netherlands, almost to the total exclusion of the

single stage procedure, but the above findings are effecting a switch to single stage testing. Stages

were ended before reaching peak stress, sometimes already after 2% axial strain. The Dutch triaxial

standard requires plotting of mobilised friction angle and cohesion as a function of axial or shear strain,

and even though the multi-stage procedure reduces strength, ϕ' and c' are chosen at some low value

of ϵ_{ax} such as 2% or 5%. Even these values are sometimes further reduced before use in stability

calculations. It will be clear that these reductions in effect serve to limit the deformations - it is the

consequence of applying limit equilibrium methods to a soil which requires large deformations to develop

strength. Stability analysis has always followed the drained method in the Netherlands. Very little empirical

knowledge has been developed with regard to s_u values. This too is changing, although much work still

needs to be done to learn to cope with the characteristics of peats and organic clays.

6.7 Triaxial extension tests and stress-induced anisotropy

The CAU triaxial extension tests on normally consolidated samples yield s_u / σ'_{vc} ratio's which also increase

as density decreases, but they are much lower than the compression values. Figure 33a illustrates the

principle of the development of the effective stress paths and the undrained strength in organic soils.

The effective stress strength envelope is high, but so also is the normally consolidated K_0 line because of

the low values of K_{nc} in these soils. Due to the high compression envelope the ratio is high in compression:

due to the high K_0 line and great distance to the extension envelope, the ratio is low in extension. In

practice however the extension paths tend to bend off and reduce the build-up of pore water thereby

limiting the decrease of s_u / σ'_{vc} . Ohta et al. (1985) provided expressions for the ratio s_u / σ'_{vc} for a number of test types including triaxial

compression and extension. From their expressions, Ladd's (1999) "degree of isotropy" factor can be

derived as

α [°]	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
angle of slip surface from the horizontal	0	15	30	45	60	75	90	105	120	135	150
τ / σ'_{v0} average	0.00	0.15	0.30	0.42	0.50	0.55	0.58	0.60	0.61	0.62	0.63
τ / σ'_{v0} active D: direct shear	0.00	0.13	0.26	0.38	0.45	0.50	0.53	0.55	0.56	0.57	0.58
τ / σ'_{v0} passive average: on slip surface	0.00	0.13	0.26	0.38	0.45	0.50	0.53	0.55	0.56	0.57	0.58

σ'_{v0} A P K o n.c. s'_u

t I

(a) (b)

Figure 33. Initial-stress-induced anisotropy in soils with high ϕ' and low K_{nc} . (a) Principle, revealing large

anisotropy: $|t_A| \gg |t_B|$. (b) Ohta et al's equations for undrained strength anisotropy applied to highly organic clay.

where

Ladd showed that in non-organic clay, K_s increases with plasticity, from approximately 0.35 at $I_p = 8.5\%$

to 0.92 at $I_p = 75\%$. The plasticity of the Dutch organic clays can greatly surpass 100%, but with

realistic parameter values, e.g. $\phi' = 50^\circ$, $\kappa/\lambda = 0.1$, and $K_0 = 0.25$, Ohta's expression yields $\beta = 0.58$ and

$K_s = 0.31$. This indicates a large degree of anisotropy and is a consequence of the high ϕ' and the low

K_0 , as illustrated in Figure 33a. Using these parameter values to further examine Ohta's expressions for τ/σ'_{vc} along a potential slip

surface, Figure 33b, values are found of 0.42 for the active point, 0.23 for the simple shear state, and 0.13

for the passive state. The value of 0.42 agrees well enough with Figure 32 at a density of say 1.35 t/m³, but

the passive value is much lower than measured. Ohta's expressions therefore underestimate the passive

resistance. Triaxial extension tests pose difficulties in execution and interpretation, among which are

avoiding or correcting for the potentially large tensile resistance of the latex membranes and filter paper

drains, the formation of necks (often near the upper platen), the effects of platen friction etc. Extension

tests on peat often pass through the states of $\sigma_v = 0$ and later on $\sigma'_v = 0$ (with σ_{rad} constant) without

showing any marked effects on the effective stress path. This must be due to tensile resistance of either

the membrane or the filter paper drains, and such tests must be rejected. Very little reliable extension

test data is available, and the prediction using Ohta of lower s_u/σ'_{vc} in extension may well be valid. On

the other hand, note that the average value of s_u/σ'_{vc}

along the slip surface between the active and passive

points is 0.26. The mobilised value would be some 11% lower or 0.22 if, following Mesri (2001), the

higher rate of strain in the laboratory tests relative to in-situ failures is accounted for. Mesri predicted

a mobilised value of 0.26 for organic soft clays and silts. This would be better in line with Figure 32 at

$$p = 1.35 \text{ t/m}^3$$

7 IN-SITU TESTS

Field tests in peats are not well developed in the Netherlands. The emphasis has always been on CPT

testing to determine depth and strength of the underlying Pleistocene sand in which piles are founded.

Only the friction ratio $R_f = q_f / q_c$ where q_f is friction value and q_c is tip resistance, is used in the

Holocene layers, to discern between peat, (organic) clay and sand. The values of q_c in peat are too low

to be read on normal scale CPT plots. The Field Vane Test is used only sporadically; the high degree of

soil variability, the large interval between readings, and the lack of interest in undrained strength values

being obvious causes of its impopularity. When s_u is needed, it is often found by dividing q_c by a factor 0.24
6 8 10 12 1.0 1.2 1.4 1.6 p [-] d e p t h b e l o w N . A .
P . [m] 0.0 1.0 q_c [MPa] 0.00 0.05 q_f [MPa] 10 100 σ'_p
[kPa] $CRS \ q_c/7.5$ -- $qc/6.5$ 0.8 R_f [%] 4

Figure 34. CPT, bulk density and yield stress profiles, Sliedrecht, km. 11.7, Betuwe railway. (outside fill).

of approximately 15. The overburden correction which is often used in the correlation of q_c and s_u , e.g.

$s_u = (q_c - \sigma_{v0})/N_k$, can amount to 50% in soft soils (e.g. in Figure 34 this is the average value between 6

and 11 m - N.A.P.). This is too high a correction to be able to use with confidence and direct correlation

with q_c is therefore preferred. Another much-used correction to q_c , for the water pressure in the gap just

behind the cone, is largely countered by side friction between the base of the cone and the gap, as shown

by Mesri (2001), and its necessity too is therefore limited. In Figure 34, results of a CPT test from the Sliedrecht site are shown together with a bulk density

profile and estimations of yield stress σ'_p . The densities were found by weighing all 20 cm sections of

a Begemann boring. The measured σ'_p values were obtained from CRS tests in a K $\emptyset$ oedometer. The

densities allow to distinguish soil types from Table 1. There is approximately 3 m of peat, between 2.5

and 5.5 m - N.A.P. This material is wood peat. It overlays organic clay, and a thin basal peat layer of

some 0.3 cm is present at 11.3 m - N.A.P. The surface layer is a slightly organic clay, and is some 0.6 cm

thick. The q_c values in the soft layers are very low. In the organic clay, it is only between 0.12 and 0.28 MPa.

In the peat, higher values occur, probably due to wood remnants. The friction ratio R_f is between 5%

and 8% in the peat (lower in the wood remnants) which agrees with the range usually taken to indicate

peat. In the organic clay below it, $R_f \approx 2.5\%$ which is quite low and indicates the material is silty. More

usual values are 3-4%. Simple correlation of q_c with σ'_p yields reasonable results, see the last profile in Figure 34, where

These correlations actually perform better than those based on $q_t - \sigma_{v0}$. For the record however, Mesri's

(2001) equation for organic soft clays and silts, given by

was also tested. q_t is the end bearing capacity and is determined from q_c by correcting for gap pore

pressure and side friction between the cone and the gap.
Taking q_t equal to q_c , Equation (18) performed

reasonably well in the organic clay below 5.5 m - N.A.P.,
but not as satisfactorily as that shown in

Figure 34. In the peat above 5.5 m - N.A.P., the factor
4.24 had to be replaced by 7.8 to obtain reasonable

correlation. The factor consists of the multiplication of N_c
(= 9), a rate of strain correction between the

fast failure in the CPT and the slower failure in the
triaxial test (= 1.24), and the factor $s_u(TC)/\sigma'_p$, and

assumes that the failure modes in both tests are similar.
In peats, as Figure 29 shows, the latter factor is

~0.7, and using this to replace the 0.38 used by Mesri for
organic soft soil, the factor 7.8 is found.

That is, the graphical procedure in Figure 28 to determine
 s_u in peat on the TCO, can be brought into

reasonable agreement with CPT q_c values (albeit via σ'_p)
, indicating that perhaps this value of s_u does

indeed determine failure conditions around the cone tip.
Most CPT soundings are performed to the Class 2 standard of
the Dutch norm which requires q_c to

be measured within an accuracy of 250 kPa in soft soils.
Maximum zero shift during a sounding was

in the order of 250 kPa using older data acquisition
systems, and it formed a major component of the

inaccuracy. This has been reduced to less than 25 kPa in
newer systems. Given further improvements in

accuracy, it will be possible to determine s_u with
sufficient reliability from q_c and perhaps also from q_f .

8 ENGINEERING PROBLEMS

Typical engineering problems posed by peats and very
organic clays and silts are due to their lightness,

compressibility and creep propensity, low strength and
stiffness and weatherability. The problems asso

ciated with low strength and stiffness in embankments and excavations will be evident. The lightness of

peat leads to the phenomenon of uplift. When dykes along the major rivers are subjected to high river

levels, uplift can occur of the soft soil on the downstream, polder side. The polder surface layers consist of

peat and soft organic clay and are pressed upwards by the high potential head which communicates from

the river through the underlying sand layers, thereby reducing the passive resistance of the inner slope.

Large berms are often needed to resist uplift. Given rising sea and river levels and continuing subsidence

of polders, uplift will become an increasing engineering problem in future. Alternative methods of dyke

strengthening are being sought e.g. deep mixing, short buried sheet piles used as dowels, nailing etc.

The lightness of peat also is problematic when tunnels are passed through it, as the buoyancy of the

tunnel finds insufficient counterweight from the soft soil. This can be partly remedied by prior deep

mixing of the peat with cement. This remedy at the same time gives the tunnel more lateral support.

A major problem is the long term creep of infrastructure constructed over peat and organic soil. Pre

compression is used on a large scale to reduce service life settlements. Vertical drainage is indispensable to

obtain economically short pre-compression periods. Even so, embankment filling usually requires staged

construction. Various enforced drainage methods utilizing vacuum techniques have been developed

to speed up pre-compression further, e.g. Beaudrain and IFCO drainage walls. Ground improvement

methods such as stone columns are used only sporadically.

A case of a failed polder dyke is now presented to illustrate some of the more salient problems

associated with peat.

8.1 The Wilnis case

In August 2003 a canal dyke at Wilnis consisting mainly of peat, failed and the downstream polder was

partly inundated, Figure 35. Surprisingly the low strength of the peat was not a factor in the failure, but

rather its lightness and susceptibility to drying. Some aspects of the failure remain a subject of research. Figure 36 shows a cross section of the dyke. This dyke consists of sedentary peat up to a level of 3 m -

N.A.P., above which anthropogenic peat was filled. The dykes which line the raised polder canals often

consist of peat, be it original sedentary material, dispersed and resedimented peat, or (occasionally,)

anthropogenic material. These dykes must permanently resist a water head of several meters. Failures

and near-failures have occurred in the past due to weathering and decomposition of the peat, and after

heavy rainfall, but also as in this case, after a long period of dry weather.

The body of the dyke is formed by soft peat and organic clay layers, which lie on the Pleistocene sand.

The crest level of the dyke was 1.5 m - N.A.P., the canal level is at 2.15 m - N.A.P., the polder ditch level

is approximately 6.4 m - N.A.P. From the canal bottom downwards, potential head reduces through the

soft impermeable layers to that in the underlying sand. It is possible that the wooden sheet pile along the

canal bank, allowed the canal level to propagate to the toe of the sheet pile. Forensic studies (Bezuijen et al. 2005) led to the following reconstruction of events. At the toe

of

the dyke near the polder ditch, vertical equilibrium of the soft layers had always been marginal due

to the potential head of the underlying sand which is fed from the canal. This is estimated at 5.8 m -

N.A.P. below the canal, reducing to 6.0 m - N.A.P. below the polder ditch, During the exceptionally

dry summer of 2003, the marginal uplift stability of the toe was exacerbated by the weight reduction

Figure 35. Wilnis dyke the morning after. 1.5 m 2.15 m 6.4 m 5.8 m wooden sheetpile anchor wall 1v:5h 1v:10h 1v:3h peat silty loamy sand pleistocene sand basal peat very peaty clay 40 m

Figure 36. Cross section of the Wilnis dyke.

of the downstream slope by evapotranspiration. Lateral movement was induced which allowed the gap

along the wooden sheet pile to widen. The gaps were possibly enhanced by shrinkage of the crest of the

dyke. Finally, hydraulic fracture occurred from the tip of the wooden sheet pile to the underlying sand.

A silty/loamy layer in the sand lying 0.5 m under the soft dyke material enabled the hydraulic pressure

from the canal to be channelled under the dyke, increasing the uplift pressure to the point that all lateral

shear resistance along the base of the soft layers was lost and the dyke failed by lateral displacement. The

failure occurred along a dyke section of some 60 m length and displaced the dyke laterally by 5-8 m. Other secondary causes were not excluded from contributing to the failure. Notably, the occurrence

of gas in the basal peat and possibly the adjoining pleistocene sand, reducing the bulk density of those

soils, and short-circuiting of water-regimes by buried conduits, cables and sand-filled ditches in the crest

of the dyke were mentioned. It is of interest to study the geotechnical properties of the basal peat at the interface with the underlying

sand. Samples were gathered from this layer some 700 m further along the dyke. It occurs between approx.

8.0-8.65 m - N.A.P. (0.5 m deeper at the failure site) and is about 0.5 m thick. It is bounded upwards by

an impermeable very peaty layer of clay. Near the crest it is at a depth of 6.5 m; near the toe at only 2.2 m. σ'_v [kPa] e H [] slope b Δe $H / \Delta \ln \sigma'_v$ ' θ 0.36 1.7 m θ G.L. 6.8 m θ G.L. 4.8 m θ G.L.. 3.8 m θ G.L. 2.8 m θ G.L. σ'_p θ 15 kPa

Figure 37. CRS oedometer test results on basal peat layer at various distances from the crest, Wilnis.

Table 2. Properties of basal peat layer, Wilnis canal dyke. Estimations G.L. Depth p_w θ σ'_p σ'_p σ'_p σ'_v

Boring nr. (m) - N.A.P. (m) - G.L. (t/m³) (%) (kPa) e θ (kPa) (kPa)

Oost-2-04 5.69 1.67 0.97 379* 15 12.5 17 18.5

Oost-2-03 5.37 2.77 1.03 624 31 10 30 31

Oost-3-3BA 4.85 3.8 1.01 511 40 8.5 45 42.5

Oost-3-2BA 3.47 4.75 1.04 455 50 8 53 54

Oost-3-1BA 1.77 6.8 1.05 450 68 7.8 56 71

* Anomalously low, probably due to loss of water from large voids. Replaced by 800% in estimations.

Its Loss on Ignition value $N \approx 75-87\%$. CRS K_0 oedometer tests and consolidated undrained simple

shear tests were performed on samples from this layer which were spaced at various distances from the

crest towards the polder ditch. Figure 37 shows the oedometer test results. The depth below ground level is an indication of the

distance from the crest, a deeper level lying closer to the crest. It is seen that the virgin slope is equal

for all samples (slope $b = 0.36$) and that the yield stress increases with depth below G.L. A check on the

yield stresses is obtained from the water content of the samples and by assuming a value of v_1 . In the

correlation in Figure 21a, $v_1 = 37.5$ emerges as a reasonable value. The water contents of the samples

are given in Table 2, and estimations of e_0 are obtained from Figure 15. v_1 and b together determine the

position of the 1-day compression curve in stress strain space and the yield stress can then be estimated

as the stress on this line at the in-situ value of v_0 (i.e., $1 + e_0$). Table 2 shows that reasonable agreement

exists with the measured values. It can be concluded that the entire layer is well characterised by one

value of v_1 and b , and that compression in the layer has been larger closer to the crest and is smaller

going towards the toe. An estimate of the total vertical stress σ_v is also given in Table 2, and Figure 38

shows that the yield stress almost equals the total vertical stress along the entire layer. Figure 39 shows simple shear (constant height) test results of this layer, where it is seen that closer

proximity to the crest results in higher shear resistance. This agrees with lower water content and higher

yield stress closer to the crest. A test on the underlying sand layer is also shown: its undrained strength

is initially not higher than that of the basal peat, but as shear strain develops and dilation occurs the sand

gathers strength well in excess of the peat

The correlation of σ'_p and the total vertical stress σ_v is surprising. It can not be explained by low

potential head of the ground water in the basal peat. Potential head is 5.8 m - N.A.P. or higher and the

polder ditch level at the time of failure is estimated at 6.4 m - N.A.P. or higher. The pressure head was

estimated to be 30 to 40 kPa throughout the basal peat layer, and this leaves a large gap between $\sigma'_{p\text{ and}}$

σ'_v . Near the toe, σ'_v is near zero. 0 200 400 600 800
 02468 depth of basal peat sample [m - G.L.] 0 10 20 30 40
 50 60 70 80 σ'_p , σ'_v [kPa] water content total vert.
 stress yield stress further from crest w_0 [%]

Figure 38. σ'_p , σ'_v and w_0 in basal peat layer at various distances from the crest, Wilnis. 10 20 30 kPa 10 20 σ'_v τ_h 4.8 m - G.L. 3.3 m - G.L. 2.4 m - G.L. underlying sand

Figure 39. Simple shear tests on basal peat layer at various distances from the crest, Wilnis. Undissolved gas exists in the pores of the peat, the basal layer included. Gas was visually detected

by a CPT camera sounding: in Figure 40 a number of gas bubbles are visible. The source of this gas

is most probably decomposition of the basal peat itself. Large gas bubbles could form underneath the

overlying impermeable peaty clay layer, with as limiting gas pressure the overburden stress i.e. the total

vertical stress σ'_v . The underlying material could be consolidated by this gas pressure, resulting in the

lower water content, higher yield stress and higher shear strength as determined above. Other mechanisms can be devised to describe the results shown here, but these will very likely also

include gas formation up to the limiting overburden pressure. It is then conceivable that low effective

stresses are permanently present or recur frequently in the basal peat layer and perhaps in the underlying

sand layer, due to gas bubbles waiting to escape. The effect may be to destabilise the dyke, curiously

however also compressing and strengthening the basal peat. Higher in the dyke this phenomenon could

be repeated where-ever peat is covered by an impermeable layer. Whether the shear resistance of the

base of the dyke can be seriously compromised remains uncertain. Further research on the influence of

gas formation at the base of dykes on their stability is of paramount importance. The drying of the slopes of the dyke was a determining aspect of the failure. Evapotranspiration

lowered the phreatic level in the dyke, causing weight loss, fissuring and shrinkage. The permeability is

greatly affected by the formation of fissures. Samples were taken close to the site of the failure at short

depth intervals to determine water content and bulk density. Surface level was 2.2 m - N.A.P. Figure 41

shows the profiles. Water content is low in the upper 50 cm, and then increases with depth. Clayey layers 5 mm

Figure 40. Gas bubbles detected by camera sounding, basal peat layer, Wilnis. 0.0 0.2 0.4 0.6 0.8 1.0 1.2 1.4 2 4
depth [m - N.A.P.] ρ , ρ_d [] 0 200 400 600 800 1000 1200
1400 w [%] wet density dry density w 0 3 5

Figure 41. Profiles of bulk density (dry and wet) and water content in Wilnis dyke shortly after failure.

can be identified where the water content is locally lower. Saturation is estimated to be above 95% below

the upper 50 cm, and this indicates shrinkage is occurring without loss of saturation. The bulk density

remains near 1. Nevertheless, weight loss occurs due to the lowering of the surface level by shrinkage.

At the toe, where vertical stability was already marginal, this induced or enhanced uplift, and as already

explained, it is thought this triggered the failure.

The upper 50 cm is heavily dried. Similar strongly dried material was found to depths of 1 m in other

borehole samples taken close to the failure zone. The degree of saturation of this material is low, and

as drying progresses, bulk density decreases but dry density increases. The latter reaches a value of

approximately 0.5 t/m^3 in heavily dried material.

The failure at Wilnis was of the horizontal sliding type. The sheet pile wall cut through the peat in the

active zone to a depth of 6 m - N.A.P. This leaves very little depth for tensile resistance to be developed

in the peat, aggravating the situation. It is conceivable that tensile strength could improve stability by

some 10%, even if it is only 2 kPa as found earlier.

9 CHALLENGES

Peat is a difficult material to deal with both in design and engineering. Its composite character implies

a geotechnical behaviour which cannot be readily modelled and captured in tests, and its disposition to

decompose with time adds a dimension to the difficulties. It is particularly unfortunate that the triaxial test

is of little use in peat due to the tension cut-off problems. Existing design methods in the Netherlands

at least, based as they are on triaxial tests and the older Dutch Cell test, are inadequate for dealing

with peat, and consequently have of necessity been geared to be overly conservative. It is necessary to

unravel the separate contribution of fibres and matrix to the constitutive response and to account for

these in material models. The "axial shear device" proposed by Molenkamp (1998), will allow this, and

Teunissen (1995) is an example of such an approach using the finite element method. This approach

may eventually make it possible to adequately deal with the high shear strength and low tensile strength,

the low stiffness, and the large anisotropy of this material. The hollow cylinder apparatus has greatly

improved the understanding of the constitutive behaviour of sands, silts and clays, but it is questionable

whether it will be useful in a material such as coarse fibrous and clayey peat. Disturbance by trimming is

difficult to avoid, and cutting through large fibres cannot be avoided. Oblique orientations of the stress

tensor may tend to rotate fibres, most likely in a non-homogeneous fashion. The length and width of fibres in commonly occurring peats easily exceed the dimensions of samples

such as are accepted by standard laboratory apparatus. Nevertheless, most testing of peat continues

to be on standard-sized samples, and insufficient attention is given to the effects on behaviour. Sample

quality also must receive more attention. Much peat occurs at relatively small depth, making high quality

extraction of large samples feasible. Large samples would be required for use in the hollow cylinder

apparatus. Due to its strong anisotropic structure, peat has a disposition towards horizontal sliding. Therefore,

the simple shear and direct shear test are useful in peat, and this fact should be exploited more fully to

develop e.g. relationships between undrained shear strength, pre-consolidation pressure and pre-shear

stress. Sample size effects in these tests, allowing extrapolation to the field scale, should be established. The Wilnis experience as analysed in this paper, has exposed a serious lack of insight in the effects of

gas in peat and other soils, which hampers a proper evaluation of the stability of dykes and embankments.

Genesis, origin and passageways of gas through the soil, its age and composition, whether dissolved,

occluded or free, size, relative volume and degree of saturation, pressure and surface tension all need

investigation. Gas may increase significantly increase yield stress and shear strength, as shown in this

paper with regard to the basal peat layer, at the same time possibly compromising stability. Gas in peat may be occluded in tiny bubbles in cell tissue, effectively giving the solids material

a large compressibility. Then, deviations might occur from Terzaghi's effective stress principle. Such

conclusions have been drawn for municipal solid waste and it is to be expected that the same holds for

peat. E.g., it may prove necessary to redefine effective stress, and to re-interpret the measured behaviour

of peat. Field measurement of peat strength has not been well developed in the Netherlands. CPT tests tend

to apply a triaxial compression mode of shearing, with the attendant tension cut-off problems probably

also occurring around the tip. Correlation with laboratory strengths might therefore be expected to be

poor, but the Sliedrecht case has indicated indirectly that the determining value of s_u in both triaxial

compression and the CPT test, is as shown in Figure 28. Direct correlations of s_u values from various

laboratory and field tests with cone tip resistance is necessary to substantiate this. In the meantime,

it may well prove that, as also indicated by the Sliedrecht case, sharper correlations may be obtained

with tip resistance q_c rather than with variously corrected cone resistance values. Given the increasing

accuracy of cone readings, it could be taken as a challenge to develop the CPT into a quick, cheap and

reliable field investigation tool in peat, together with reliable correlations for undrained strength, yield

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Characterization of Escuminac peat and construction

on peatland

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ABSTRACT: Peat could be any material between two such extremes as a coarse-fibrous matting and

a jelly-like gyttja. It follows that it is extremely important to describe the peat in detail. A case record

or specification for construction on peatland is of little value without such detailed description. In a

typical sedentary peat, most of the primary compression takes place during construction. Interparticle

and intraparticle waters in the peat are expelled simultaneously because the peat structure is open

at all common degrees of humification. Thin road embankments consisting of well compacted fill and

underlain by corduroy provide a better bearing surface than even much thicker embankments underlain by

flexible fabrics. Field consolidation is considerably faster than laboratory consolidation. The main reason

for this is thought to be that drainage in the field is generally three-dimensional because the unloaded peat

adjacent to the embankment readily separates vertically due to horizontal hydraulic fracturing. Most, if

not all, failures of road embankments on peatland have been found to be due largely to the existence of

a very soft mineral soil layer underlying the peat.

1 INTRODUCTION

1.1 Definition of Peat

Peat is defined (Muskeg Engineering Handbook 1969) as "a component of organic terrain consisting

of more or less fragmented remains of vegetal matter sequentially deposited and fossilized". Organic

terrain is defined (loc. cit.) as "a tract of country comprising a surficial layer of living vegetation and a

sublayer of peat or fossilized plant detritus of any depth, existing in association with various hydrological

conditions and underlying mineral formations."

The first definition given above (Muskeg Engineering Handbook 1969) does not include any inorganic

(e.g. pp. 123-126) pertain to soils described as peaty soil, organic soils, muck, peat soils, marsh and

marshland deposits, peat land, coastal marsh peat/muck/clay, muskeg, peaty alluvia, soft peaty ground,

(generally referred to as the "ash" content) in some of these soils is as high as about 80%. This is clearly

not consistent with the original definition, which suggests that peat contains little or no inorganic matter.

The following two paragraphs are quoted from Wetland Soils (Richardson & Vepraskas, 2001): "Many terms have been used to describe organic soils. The term "peat" has been used for many years

as a general term to describe the soils with various amounts of un-decomposed plant remains. Early

emphasis was placed on the botanical source of the plant remains, and several types of peats (limnic

peats, telmatic peats, terrestic peats, etc.) were recognized." "Specific terms have been defined for organic soils and organic material. The majority of the terms

depend on the degree of decomposition of the organic matter. These terms are used throughout the world.

In Alaska and other northern areas of North America, as well as areas in Europe with large expanses of

organic materials, peat is still used today as a general textural term indicating a high fibre content. The

term "peatland" is used as a general landscape term to describe the expansive area of their occurrence.

Peatlands form in depressions, slopes and raised bogs and occur over a range of climatic conditions from

boreal to arctic. Peatlands are often divided into bogs, fens, mires, and swamps. These are particular

landscapes or ecosystems and are often given the general description of peat-accumulating wetlands.

Bogs are areas of peat that are acid. Bogs are formed primarily from shrub or moss vegetation. The tannin

content of bogs is high. Fens are sedge and grass-like plant dominated areas of organic materials that

contain considerable bases, such as calcium. Swamps are formed under woody vegetation with variable

amounts of tannin. Mires is a common but confusing term used in Europe to indicate the ecosystems in

which waterlogged peat has accumulated, usually in raised areas. Mitsch and Gosselink (1993) suggest

that mire, as used in Europe, is synonymous with any peat-accumulating wetland." Stanek (1980) has suggested the following definition of peat: "Generally unconsolidated organic material consisting largely of organic residues accumulated as a

result of incomplete decomposition of dead plant constituents under conditions of excessive moisture

(submergence in water and/or water-logging). Note (1) May contain a very variable proportion of transported mineral material. Note (2) May form in both base-poor and base-rich conditions, and either in situ or from accumulations

of water-borne organic material, as on lake bottoms. Note (3) May largely consist of or may contain layers of coprogenic elements and comminuted plant

remains (such as gyttja) or humus gels (such as dy)." The work presented in this paper pertains to peats of low or negligible mineral content. Most of the

field and laboratory work was carried out on a sphagnum peat of mineral content 1-2%, i.e. 98-99%

organic content.

1.2 Brief history of construction on peat

Dewar (1962) has described roads which were built on peatland in various parts of England as early as

about 2500 B.C. They were made with wedges and stone (or bronze) axes in the new stone (neolithic)

age (or in the bronze age) and consisted of sleepers or corduroy pegged and held down by stringers.

The distance between the sleepers varied from zero to as much as 1.5 m. In places these roads were

covered, at the date of discovery (1834 to 1962), with over 2 m of sphagnum peat, which had formed

there during the last 3000 to 4500 years. Apparently, the trackways were submerged reasonably soon

after their construction and so preserved without being broken up or rotten. It is held (Muskeg Engineering Handbook 1969, p. 150) that very few changes in road construction on

peat took place until the current century. That this is not so is amply demonstrated in a paper presented

to the Institution of Civil Engineers of Ireland in February 1846 by B. Mullins, C.E., Vice-President of

the Institute of Civil Engineers of Ireland, and M.B. Mullins, A.M., C.E., Member (Mullins & Mullins

1848). This paper reveals a thorough knowledge of the origin of peat and a thorough technical insight

into peat mechanics. Considering the advanced concepts and techniques used by Mullins and Mullins, it is in fact not

possible to trace any further major developments in peat construction techniques to the present day.

Although peat drainage is a common method in agriculture and peat production, deep drawdown by

gradual drainage such as that used by Mullins and Mullins does not appear to have been used anywhere

in the twentieth century (MacKenzie 1904, Hanrahan 1952,1967, Tresidder & Fraser 1955, Tresidder

1954, 1958, Brownridge 1957, Root 1958, MacFarlane 1956, Warnock 1967, Muskeg Engineering

Handbook 1969). The use of corduroy or fascines was, however, quite common. MacKenzie (1904)

recommends "draining and side ditching where possible, thus making bog firm enough to carry, in

preference to cross logging". The use of lightweight fill, first conceived by Mullins & Mullins (1848) in the form of compacted peat

(preferably dried), was also recommended by MacKenzie (1904), who, as Chief Engineer Inter-Colonial

Railway at the time of his lecture, clearly had been involved in construction well before the turn of the

20th century. MacKenzie recommended that banks crossing a morass, peat-bog or swamp be made "as

light as possible, using turf, peat, sawdust or cinders". The use of compressed peat bales as lightweight fill on peat was probably introduced in Holland

around 1935 (Alexander 1938), although similar fill had been used as a mat under railroads in Norway

from 1929 (Skaven-Haug 1945). The use of corduroy or fascines was, as pointed out above, used in Ireland and Holland before 1846.

Fascine mattresses were perfected in Holland where gravity drainage can not be used and where there are

large areas of soft peatland. Corduroy construction is also found at Point Escuminac, New Brunswick,

where a 0.4 to 0.5 m bank was constructed on transverse logs, 4 to 12 cm diameter (no spacing), underlain

by three longitudinal stringers. Part of this road was

constructed some time before the turn of the 19th

century and part of it later. It is of interest to note that each log had been felled by two blows of a

broad-axe. Concrete raft construction was used in Ireland during the late 1920's and early 1930's (Warnock 1967).

Most of these concrete slab roads were still in service in 1967, carrying traffic of a volume and intensity

much higher than what they had been designed for.

There seems to be general agreement that the "floating" of roads on peat is a method to be used for

secondary roads only. However, Samson & La Rochelle (1972) describe the design and construction

of sections of an expressway between Quebec City and Montreal where it crosses peat of 3.0 to 5.8 m

depth. Preloading techniques were used throughout and this method of construction proved to be quite

successful. In conclusion, it may be stated that there have been few, if any, developments in the construction of

roads on peat since that practised by Mullins & Mullins (1848) about 160 years ago. In fact, their methods

seem rather superior to those practised today. One reason for this is perhaps that modern machinery and

the present-day pace do not allow sufficient time for preparatory work, such as long-term compression

by drawdown and loading.

1.3 Engineering aspects of peat formation

It is not the purpose of this paper to explore the very complex biological conditions involved in the

formation of peat. It is, however, of interest to briefly examine the conditions governing growth and

decomposition of peat deposits. It is estimated that about 95% of all sedentary peat deposits have been formed chiefly from plants

growing under aerobic conditions (Korpijaakko 1977). Once the accumulation of peat has commenced,

the high water-holding capacity of peat maintains a surplus of water, thus ensuring continued accumu

lation, provided that the rate of accumulation exceeds that of decomposition. The rate of decomposition

also depends on the environment. It is relatively rapid under aerobic conditions, but is slowed down

several thousandfold under anaerobic conditions (cool climate and surplus of water). Thus aerobic con

ditions are conducive both to growth and to decomposition, while anaerobic conditions arrest surface

growth and slow down subsurface decomposition. These apparently contradictory conditions lead to a

complex pattern of intermixed growth and decomposition of moss plants, sedge plants and other peat

forming plants. Since sedge plants are much more resistant to decomposition, a natural reinforcement

(fibres) is thus provided where it is needed most, i.e. in decomposed moss peats.

Another source of reinforcement is provided by the living mat. Every part of the peat deposit started

its existence at the growing surface as part of this mat. From observations of present-day peat deposits, it

would seem safe to assume that the mat was in general composed of a mixture of moss and fibrous plants,

and often shrubs and trees. Again, therefore, a reinforcing mesh would be provided by natural means,

scattered throughout the deposit depending on the climatic conditions prevailing at various periods. Since every part of a raised bog peat deposit started its existence at the surface, every part of the deposit

has also been exposed to weathering such as drying, wetting, snow cover and frost. All of these agents

would have a profound effect on the mode of formation of the deposit, particularly on the directly exposed

mat and on the partly saturated and therefore readily compressible zone immediately below the mat. For

example, a snow drift of 2 m thickness could exert a pressure on the deposit of about 8 kPa. Although

this is a relatively low pressure, it would cause considerable compression of the deposit, particularly in

the upper non-saturated portion. The effect of drying could produce even higher compressive loads, e.g.

from capillary stresses caused by drying or simply as a result of decreasing pore pressures (increasing

effective stresses) during ground water drawdown.

The unit weight of peat may be less than that of water, and since the ground water level in a peat

deposit is generally close to the surface, the effective overburden pressure is generally negligible. Yet

the porosity with respect to interparticle voids only may be as low as or even lower than that of loose

sand. There can be no doubt that this relatively low porosity is a result of the stress history of the peat,

as outlined above.

1.4 Peat fabric and structure

Microscopic examinations as well as counts and weighings of several samples of SH 2-3 peat (see Appendix

A) from Escuminac showed that this peat consisted in general of 10-15% stems, 85-90% leaves and leaf

fragments, and less than 10% of elements smaller than 0.074 mm. It was found that the water content

of individual sphagnum leaves was in the range 900-1100%. The water content of sphagnum stems was

determined to be in the order of 700%. Figures 1-20 are

selected SEM (scanning electron microscope) micrographs of sphagnum peat.

1.4.1 Leaf and stem structures

Sphagnum leaves consist of hollow hyaline cells and chlorophyllose cells, alternating across the width

of the leaf. The thickness of a leaf corresponds to the height of one hyaline cell (Fig. 4). The hyaline cells

are reinforced with several baffle walls in each cell (Fig. 5). There are several relatively large openings

in each hyaline cell on the upper surface of the living (H 1) leaf (Figs. 1-3) and a few small openings in

the lower (bottom) part (Fig. 2). At the H 1 stage some of the openings are covered with a thin membrane,

which has a single opening near the edge (Pheeney 1976). Sphagnum stems consist of hollow cells. The outer membrane of the living (H 1) stem is not perforated

(Figs. 6-7). The cells are tubular in shape and appear to be separated from each other along the length of

the stem by end membranes. No openings have been observed in the cell walls or in the end membranes

of living stems. The thickness of the stems varies within wide limits.

1.4.2 Decomposition (Humification)

The process of decomposition (humification) of sphagnum moss may be assessed by examining sphag

num peat samples at various stages of humification. It is not possible to state categorically the exact

degree of humification of any one sample, but for the purpose of examining the effect of decomposition

on the plant it is sufficient to consider certain common ranges, such as H 1-2 , H 3-5 , H 6-8 and H 9-10 . In the

range H 1-2 both leaves and stems appear more or less as described in the previous section. In the range

H 3-5 the membranes of both sides of the leaves are at least partly decomposed, leaving a relatively open

framework of cell walls (Figs. 9-14). The stems in the range H 3-5 appear to have lost their core cells

Figures 1-4. Figure 1. Sphagnum (SH 1) apical bundle (18X). Outer (upper) surface of SH 1 leaves in view. Vertical

stem not visible. Figure 2. Sphagnum leaves (SH 1) on apical bundle (80X). Detail of Figure 1. Upper surface

(membrane) of leaves is extensively perforated, lower surface (membrane) only slightly perforated. (A) Hyaline

cells, upper surface; (B) hyaline cells, lower surface (dark thin lines = baffle walls); (C) chlorophyllose cells (dark

thick lines). Figure 3. Sphagnum leaf, upper perforated surface (180X). Detail of Figure 2. Approximate diameter

of openings (A) in hyaline cells (B) is 10-15 μm . Hyaline cells are hollow and reinforced with baffle walls. Each

hyaline cell has several openings. Figure 4. Sphagnum leaf cross section (400X). Lower surface seen on top. Large

cells (A) = hyaline cells; note reinforcing baffle walls (B). Smaller cells (C) = chlorophyllose cells (alternating with

hyaline cells). Part of section struck through openings in upper membrane (bottom in this photo). Thickness of

leaf = one hyaline cell $\approx 15 \mu\text{m}$.

Figures 5-8. Figure 5. Sphagnum leaf cross section (2100X). Detail of Figure 4. Note reinforcing baffle walls

(A). Typical thickness of wall (B) between hyaline and chlorophyllose cells is approximately $0.5 \mu\text{m}$. Note openings

between cells. Figure 6. Sphagnum stem (SH 1-2) (18X). Leaf bracts (B) still attached. Stem branching near bottom

(C). Full cellular growth (A) indicates active cortex (SH 1). Stem diameter approximately $500 \mu\text{m}$ (0.5 mm). Outer

membrane (integument) not perforated at this stage. Figure 7. Sphagnum stem (180X). Detail of Figure 6. A = cells;

B = outer membrane (integument). Figure 8. Longitudinal section of stem from Sphagnum peat (in situ classification

SH 4) (100X). Most of cell membranes decomposed, leaving structure very porous. Note complete loss of core cells.

A = outer remaining cells; B = hollow interior.

Figures 9-12. Figure 9. Sphagnum peat leaves (in situ classification SH 4) (100X). Most of leave membranes

decomposed, both upper and lower sides, leaving fabric extremely porous (A). Some membranes intact (B). Figure

10. Sphagnum peat (SH 3) sampled under Escuminac road ($\sigma' N \approx 15$ kPa) (35X). Note alignment and bending of

leaves and stems, and loss of cell membranes. Figure 11. Loaded surface of SH 3 peat after 85% compression and

oven-drying in compressed state (45X). Alignment of leaves and stems evident; also partial loss of cell membranes.

Note that microstructures (leaves and stems) are otherwise reasonably intact, despite extreme compression. Figure 12.

Detail of intact leaf cell (with membranes) from Figure 11 (200X). A = mycelium fibre (fungus).

Figures 13-16. Figure 13. Loaded surface SH 3 peat after compression under 7000 kPa and oven-drying dur

ing rebound (70X). Figure 14. SH 3 peat cut from large-ring shear failure surface (oven-dried after shear) (25X).

$\sigma' N \approx 13$ kPa. Note extensive alignment of leaves and reasonably intact microstructures. Figure 15. SH 6-7 peat from

pure shear (torsion) failure surface (50X). $\sigma' N \approx 0$. Extensive alignment of microstructures in direction of shear ($\approx$ hor

izontal plane in situ). Figure 16. Fractured edge of disturbed peat sample classified in situ as SH 8 (60X). Oven-dried,

broken off with tweezers. Note large content of less decomposed peat (SH 3-6). Fabric consists of alternating layers

of amorphous-granular material (SH 10) and less decomposed material (SH 3-6). A = amorphous structure; B = leaf

and stem structure; C = leaf and stem microstructures.

more or less completely, and the outer cells of the stems have lost their membranes (Fig. 8). In the range

H 6-8 it is no longer possible to detect distinct leaf structures (Fig. 15).

The range H 8-10 would be expected to consist mostly or completely of fully decomposed material,

referred to as amorphous or amorphous-granular material (von Post 1922, Radforth 1952). However, a

sphagnum peat designated as SH 8 (Korpijaakko 1977) proved under SEM investigations to consist of

alternating layers of H 3-6 and H 10 material (Fig. 16). This type of fabric is not uncommon, as pointed

out by Radforth (1969, p. 39). It seems reasonable that a mixture of truly amorphous-granular material

(H 10) and less decomposed material (H 3-6) would behave as H 8 material when classified according to

the von Post method. A parallel situation exists in mineral soils: a typical clay material may not contain

more than 20% clay size minerals, and yet the material behaves essentially like a true clay. Little is known about the exact nature of the amorphous-granular material (H 10). The individual

elements forming the porous amorphous-granular matrix consist, in all probability, at least partly of

various organic acids. Research on organic acids (Chen et al. 1976) indicates that such acids themselves

have a sponge-like structure. Any water or other liquid within these microstructures would presumably

be chemically or otherwise firmly bound and would not therefore take part in free flow such as that

required for primary consolidation. These microstructures are presently thought of as being solids with

respect to free flow, but as being readily susceptible to plastic deformation under sustained stress.

1.4.3 Compression and shear

The effect of compression and shear on the Sphagnum peat fabric is seen in the micrographs in

Figures 10-15. In compression the structures tend to align themselves in the plane of major principal stress

(Figs. 10-12). They also appear to remain surprisingly intact, even under very considerable compression

Figures 17-20. Figure 17. Sedge sheath from peat classified as S(Er)H 8 (90X). This material is sheet-like in structure

and does not have any discernible water-holding cells.

Figure 18. Sedge stems and fibres from material classified as

CEr(S)H 7 F 2-3 R 2 (80X). Sheet-like structures are stem sheaths (integuments). A = stem cuticle; B = fibre (rhizoid).

Figure 19. Sedge stems (80X). A = stem sheath (integument); B = interior porous stem structure. Figure 20. Sedge

stems (200X). A = stem sheath (integument); B = interior porous stem structure.

(85%, Figs. 11-12). Under the extremely high load of 7000 kPa (Fig. 13) the structures are still

discernible, but distortion is evident. In shear the microstructures tend to align themselves in the plane of shear failure (Figs. 14-15). The

fabric in Figure 14 has been subjected to about the same vertical stress as that in Figure 10 and to a shear

stress of about 15 kPa. The horizontal displacement in the plane of shear failure (Fig. 14) was about

100 mm.

1.4.4 Phase diagram

Figure 21 shows a typical phase diagram for sphagnum peat, prepared on the basis of microscopic

investigations and measurements. The division of the void space into interparticle and intraparticle voids was quantified by considering

both the average water content of individual leaves and direct measurements of sections of leaves and

stems, such as those shown in Figures 4-7, at various degrees of magnification. The SEM measure

ments indicated that about 8% of the leaves consist of solids, corresponding to a porosity of 0.92. The

water content of individual leaves (900-1100%) corresponds to a porosity of 0.93-0.94, assuming full

saturation. The water content of the fibres, measured in one group of samples only ($w = 670\%$), would

correspond to a porosity of 0.91. Considering that about 85-90% of the SH 2-3 sample consists of leaves,

an average porosity of 0.92 was used for the peat elements (microstructures). It is seen from Figure 21 that only about one third of the total water in the SH 2-3 peat is located

in the interparticle voids. From the SEM investigations of compressed samples (e.g. Figs. 10-11) it is

apparent that the porosity with respect to interparticle voids only is still relatively high after compression.

Consequently, it may be concluded that the intraparticle (cell) water as well as the interparticle (voids)

water is squeezed out during compression. Because of the large openings in the intact cells and the often

completely open framework (e.g. Fig. 9) it seems clear that the expulsion of intraparticle and interparticle

water would take place simultaneously.

1.4.5 Investigations of a sedge peat

Figures 17-20 are scanning electron micrographs of a sedge peat from Shippagan, New Brunswick. This

peat was classified as CEr(S)H 7 F 2-3 R 2 (mainly Carex remnants, significant content of Eriophorum, low

content of Sphagnum, strongly decomposed, moderate to high content of fine fibres, moderate content

of coarse fibres; see Appendix A). However, the exact determination of the genera is difficult in any

sedge peat of humification degree H 7 , considering that the sedge family has about 90 genera in all.

The fibrous nature of this peat is apparent from the designation F 2-3 and R 2 and from examining the

peat under a stereo microscope (60-100X). Few, if any, leaves (except some Sphagnum leaves) could be

observed under either the stereo microscope or under SEM examination. The micrograph in Figure 18

is fairly typical of any portion of the samples investigated, except that the rhizoids are generally much

more abundant than indicated by this micrograph. Three different constituents may be distinguished in

the sedge peat: (i) the rhizoids (Fig. 18); (ii) the sheaths, or integuments (Figs. 17-20); and (iii) the

interior stem cell structure (Figs. 19-20). The third type of constituent observed in the sedge peat appeared as an amorphous-granular (Radforth

1952) material, existing randomly throughout the samples investigated. At natural water content this

material appeared as granules having a jelly-like consistency. Although the "jelly" had no distinct

Figures 21. Phase diagram for typical SH 2-3 peat from Escuminac, N.B.

structure and thus was not readily identifiable, there can

be little doubt that it consisted of a mix

ture of amorphous-granular sphagnum peat and remnants of the inner cell structures of the sedge stems

and rhizoids.

1.4.6 Water content

It has been suggested by several investigators (e.g. Boelter 1969) that the water content (w_v) of peat should

be expressed as the portion of the total volume that is occupied by water. If the peat were completely

saturated, w_v would correspond to the porosity (n). It is held that the expression of water content (w)

by weight as the ratio of the weight of water to the weight of solids is generally extremely high (500-

3000%), that these high numbers are therefore somewhat meaningless, and that w is very sensitive to

small changes in the weight of solids. However, the opposite can be said to be true for w_v , which shows

very little sensitivity to a change in structure. An example is the peat under a road at Escuminac, NB.

Its porosity ($\approx w_v$) before loading was 0.96 and after loading 0.91, i.e. a rather insignificant numerical

change in w_v despite a very conspicuous change in structure and in resistance to sampling. The water

content by dry weight (w) of this peat was 1500% before loading and 750% after. This significant change

in w reflects better the very large compression and the considerable changes in structure and resistance

as a result of loading. For geotechnical purposes therefore the expression of water content in terms of

dry weight is more useful than that in terms of total volume ($\approx n$). Water content (w) in terms of dry

weight should therefore be retained.

1.4.7 Summary (peat fabric and structure)

(1) An SH 2-3 sphagnum peat from Escuminac was found to consist of 85-90% leaves and 10-15% stems and fibres. The water content of individual leaves was found to be of the order of 1000% and that of individual stems about 700%. The hollow cells of the leaves are reinforced with baffle walls, which lend considerable strength to the leaf structure. These structures are extremely resilient and do not appear to rupture or break down under load. The leaves are extensively perforated in the H 1-2 condition and are therefore readily pervious to water flow. At H 3 and higher degrees of humification the openings in the membranes become larger as the membrane decomposes and eventually disappears, leaving an open framework structure. The same applies to the stems. About one third of the total water content is located in the interparticle (external) voids. The remainder is located in the intraparticle (internal) cell voids.

(2) The mode of formation of a raised bog peat deposit results in a horizontally aelotropic fabric interspersed with plant, shrub, or wood fibres. Horizontal or near horizontal matting is common on account of the past and present exposure of every part of the peat to compression from drying, groundwater fluctuations, and snow loads. The degree of alignment increases with increasing stresses, as evidenced by the matted fabric samples taken under road embankments.

(3) The fabric of an SH 8 peat from Shippegan, NB, was found to be stratified, consisting of alternating layers and lenses of SH 10 and SH 3 material, each constituting about one half by volume. The SH 10 material was completely amorphous.

(4) A sedge peat from New Brunswick was found to consist of mostly hollow rhizoids and integuments (stem sheaths) and an amorphous-granular material believed to be the remnants of the sedge plant cells. The cell structure of the stems could be observed in the scanning electron microscope. It was concluded that approximately one half of the water in the sedge peat was located within the peat constituents and that, on compression, this water could readily escape through the ends of the partly or completely broken structures.

(5) The expression of water content as a percentage of dry weight should be retained. The alternative expression of water content as a percentage of total volume already

exists in geotechnical nomenclature (in terms of porosity and degree of saturation), but it does not reflect the large compression normally occurring in peat.

(6) Proper identification of peat should include a description of the constituents as well as degree of humification and water content. The term amorphous-granular should be reserved for nonfibrous truly amorphous peats only, and since such peats are rare, this term should be used with some caution. The term fibrous applies to practically all peats and is therefore by itself not a sufficient description.

(7) The belief (hypothesis) that the consolidation process in "fibrous peat" involves the expulsion of the interparticle void water as primary consolidation, followed by the expulsion of the intraparticle void water as secondary consolidation, has been shown to be misleading. The expulsion of interparticle and intraparticle water takes place simultaneously. Even in peat with a low degree of humification, the openings in the organic membranes are ample for free drainage. At higher degrees of humification, normally found in peat, part or all of the membrane has decomposed, leaving a structure even more open to flow.

1.5 Classification of peats and peatlands

1.5.1 Peats

Various systems have been developed for the identification and classification of peat (von Post 1922,

Kivinen 1948, Radforth 1952, 1955, Largin 1973, Puustjärvi 1973, Korpijaakko & Woolnough 1977).

The effects of fabric and structure on the geotechnical behaviour of peat have, however, generally been

assessed indirectly on the basis of measured geotechnical properties rather than on a direct examination

of the peat fabric. A modified form of the von Post - Kivinen classification system (Kivinen 1948, Korpijaakko &

Woolnough 1977) as used in this paper is given in Appendix A. This system generally follows that used

by Korpijaakko and Woolnough, but numerical values have been assigned to water contents and fibre

contents, and the degree of humification has been presented in tabular form (Table A1) to facilitate a

comparison between different groups. Both the Radforth and the von Post classification systems take into account the contents of fibres,

amorphous-granular material, and wood and shrub remnants. In the Radforth system the three primary

elements used for classification are amorphous-granular material, fibres, and wood, with a sub-division

into fine and coarse fibres. No direct reference is made to moss or mossy elements in the 17 main

categories; these may, however, be inferred from an examination of the typical sample photographs

(Radforth 1955). In the von Post system the species designations specify the types of plant involved

(S = sphagnum, H = hypnum, Er = eriophorum, C = carex, etc.), while F designates fine fibres, R coarse

fibres, N shrubs, and W wood (Appendix A). The designation SH 3 F 2 would thus be improper since a

pure Sphagnum peat would not have the relatively high content of fine fibres indicated by F 2 . SCH 3 R 2 or

SERH 3 F 2 would indicate a moderate content of Carex and Eriophorum fibres, respectively; these are much

stronger than moss fibres and are therefore of considerable interest to the engineer. From an engineering

point of view, the stronger the fibres and the greater their number, the more reinforced is the peat. Thus

a peat with the designation CSH 3 F 2 R 2 is likely to be a much better foundation material than SH 3 F 1 . For engineering purposes, therefore, it is of advantage to know the approximate proportions of the

various constituents and types of fibres. This is even more important where amorphous-granular material

is involved, since this material is particularly weak without reinforcement. Of the various terms used in

existing classification systems, fibrous and amorphous-granular are

those used most often by engineering investigators. In the von Post system these may be represented by,

for example, ErCH 2 F 2 R 3 and (S)H 10 , respectively. Although these peat types do exist, the majority of all

peat deposits consist of moss peats. Truly amorphous-granular peats (H 9-10) are relatively rare, and they

are generally reinforced with fibres, as indicated above. A truly fibrous peat may be compared to felt or

fibreglass matting and a truly amorphous-granular peat to jelly. The author contends that peats generally

referred to in engineering literature as amorphous-granular are in fact moss peats (in some cases, they

may also be highly organic soils). The present investigation has indicated that moss peats behave like

a fibrous material and that the distinction between "fibrous" peats and moss peats is therefore one of

degree only.

1.5.2 Peatlands

Peatlands may be divided into different categories depending on for example moisture or nutrient con

ditions. The latter is the more practical and also the most common division. It also corresponds to the

division of peat types into fen (eutrophic, rich in nutrients) and bog (oligotrophic, poor in nutrients) peats.

This type of classification can also be applied to the geological product of the bog, viz. the peatland

soils, i.e. the peatland could be divided into marsh and moss peatland. If only the upper layer of the land

is of interest (e.g. for cultivation purposes), this classification is fine. However, it is not sufficient for

classification of the entire deposit and its mode of formation, since this could readily include both fen

and bog stages. Consideration of the distribution and characteristics of the moisture in the ground and the main genetic

groups (von Post 1916) has led to the following classification of peatlands:

1. Topogenic peatlands - whose formation is totally dependent on topographical conditions; they are formed in moist depressions with extra water and in stagnant waters.
2. Ombrogenic peatlands - which are developed as a result of rainwater falling on their surface; they are formed in areas of high precipitation and depend on atmospheric water as the only source of moisture.
3. Soligenic peatlands - which are created by the supply of surface runoff or by a high groundwater table (moving or percolating groundwater). In some cases topogenic primary paludification (water-logging) may be observed within or near

topogenic peatlands formed by the in-filling of ancient lakes. Thus very rarely can a peatland deposit be

categorically classified as one single type, since in most cases it is formed by a combination of different

types of peatland. It is for example common that an area starts developing as topogenic peatland and

later, when the climate has changed, continues to develop as ombrogenic or soligenic peatland.

The characteristic end product of ombrogenic paludification is a raised bog. The mode of growth of

a raised bog is more or less the same regardless of its initiation as in-filling of a lake or as primary

paludification. The growth of raised bogs is totally dependent on the influence of rain water, which is

their only source of water and nutrients. This is borne out by their extremely low content of nutrients

and by their low mineral content.

The key for the mechanism of the growth of raised bogs is their surface. This displays a mosaic of areas

of varying moisture conditions, which may vary from small water-filled ponds to quaking sphagnum

Figures 22. Boring log, test area II, Escuminac, N.B.

surfaces to dry heather or lichen-covered mounds. The small rimpis (ponds) are in the process of being

filled in and the dry mounds are small-size end stages in the development. Some rimpis stay open during this development, but the surrounding areas continue to grow. This

results in the development of deep ponds in raised bogs, having nearly vertical slopes and containing

gyttja or dy.

2 POINT ESCUMINAC, NEW BRUNSWICK, THE MAIN PEATLAND SITE CONSIDERED FOR THIS PAPER

The Escuminac peatland site covers an area of about 2×6 km at the Point Escuminac peninsula situated

at the north-east coast of New Brunswick, Canada. The peat extends about 6 km along the peninsular

coastline. A typical boring log representing a test area at Escuminac is shown in Figure 22. The raised peat bog

at Escuminac is generally referred to as a sphagnum peat deposit, but the lower part consists actually

of sphagnum-carex and carex-sphagnum peat. Eriophorum remnants, mixed with carex and sphagnum

remnants, generally are found near the bottom next to the sandy till subsoil. The upper approximately 4 m of the profile shown in Figure 22 consists of sphagnum peat interspersed

with some shrub (N) rootlets and fibres. The degree of humification generally is H 2-4 , but higher values

(H 5-6) exist at 1.0 to 1.5 m depth. The average water content, as compiled from several boreholes in

the test area, increases from about 1200% just under the 0.3 m thick mat to about 1600% at 4 m depth.

The range of water content at any depth is very high (600 to 900 percentage points), clearly showing that

water contents from one boring only would not be representative of a larger area. The mineral content

of the sphagnum peat is insignificant (1 to 2%). Below 4 m depth the sphagnum peat contains varying amounts of carex remnants. The degree of

humification generally increases with depth up to H8 and the average water content decreases corres

pondingly from 1600% to about 700% at 6.8 m depth. It is possible that the SH5-6B4F1R1 layer at

1.0-1.5 m depth is a "Weber's Grenzhorizont", a "persistent horizontal aquiclude to vertical drainage

and pore pressure relief under engineering works" (Hobbs 1986). The mineral content remains low (1 to

2%) to about 6 m depth, but increases to 3 to 4% in the CSH 8 layer and then increases sharply near the

transition to the mineral subsoil (clayey till). A thin transition layer of ErCSH 8 peat (also called ooze)

exists just above the till; the mineral content of this layer is about 20% and its water content about 500%.

Throughout its depth the Escuminac peat is locally interspersed with woody erratics. The water content

of peat (% of dry weight) was found to vary over a wide range from a few hundred per cent to greater

than 2000%, large changes occurring over small distances. For example, it was found that at any selected

depth within the 75 m by 15 m test area at Escuminac, the water content varied by at least 600 percentage

points, the total variation within the site being no less than 1600 points.

The aelotropic (banded) nature of this peat becomes apparent when handling undisturbed samples:

the peat comes apart readily in the vertical direction, but offers considerable resistance to tension in the

horizontal direction. In accordance with Taylor's (1948) definition of aelotropy, the type of aelotropy found in this peat is

one resulting during deposition or from stress applications, rather than a true stratification. Evidence of

this was provided during a program of undisturbed sampling below a road embankment on the Escuminac

peat. The compression under the embankment, which had been in existence for 10 years at the time of

sampling, had reduced the water content of the peat from 1500 to 750%. The resistance to sampling

under the road was very much higher than that adjacent to the road, and the peat had assumed a flaky

nature, consisting of relatively thin felt-like mats, which could be peeled apart readily. The compressed

mats were generally oriented in a horizontal direction. The porosity of this compressed material was still

as high as 91%, as compared with 96% before loading.

The Escuminac peat reaches a depth of up to 8 m. A corduroyed road built around 1870 stretches

across the entire length of the peatland. Portions of this road were relocated in 1967 as a result of coastal

erosion, and detailed information about the 1967 construction was available locally from the contractor.

The coastal erosion at Escuminac has produced extensive peat cliffs up to 5 m in height. The entire

cross-section of this part of the peatland can therefore be examined directly, and the eroded road can

also be examined in section and actually also from

underneath.

A new road was constructed across the Escuminac peat bog in 1967 to replace the partly eroded 1870

road. Reconnaissance of the road with the contractor revealed that the settlement of the 1967 fill had

been extremely variable. Several soft spots had been encountered, and fill in excess of 3 m thickness

was occasionally required to "float" the road, the road surface still being only at original ground level.

In other areas the peat was much firmer and very little fill was required.

This brings up an extremely important characteristic of raised bogs (as described above, 1.5.2): In

general, it cannot be assumed that peatland soils (i.e. peat) are uniform even if they appear to be (as

they certainly do at Point Escuminac) and even if the rimpis (open ponds) are avoided. According to the

contractor constructing the 1967 road, the soft spots encountered during construction was not recognized

until the fill was actually being placed.

3 SOME GEOTECHNICAL PECULIARITIES OF PEAT

3.1 Anisotropy and fibrosity

In general, peats consist of a network of peat plants interspersed with woody remains and organic remains

from other vegetal growth. The degree of decomposition of the various constituents may vary widely,

but the fabric as a whole, in all but a few rare cases, is of a fibrous nature. The fibres include leaves,

stems, leaf stalks, rootlets, rhizoids (root-like filaments), woody remains, and any other elongated plants

or plant remains. These fibres provide a reinforcement similar to that in reinforced earth, except that the

reinforcement in peat is discontinuous. Cube samples of sphagnum peat were tested in unconfined compression under a stress of 14 kPa.

Typical results are shown in Figure 23. It is seen that the lateral expansion is relatively much greater

when the stress is applied parallel to the direction of the fibres. Several cube samples were tested in

various directions, and the least lateral expansion (perpendicular to the direction of loading) always

Figures 23. Deformation of cube samples of SH 3 peat in unconfined compression. (A) Loading perpendicular to

matting. (B) Loading parallel to matting.

Figures 24. Schematic illustration of horizontal fibre reinforcement in triaxial sample.

occurred when the load was applied perpendicular to the prevailing direction of the fibres. This clearly

shows that the fibres, including the flat, elongated leaves, offer an internal resistance to expansion

through frictional forces. Figure 24 is a schematic illustration of the reinforcement provided by those fibres that are aligned

perpendicular to the major principal stress. The internal resistance of the fibres is a function of the

friction between the fibres (or between the fibres and the matrix) and of the tensile strength of the fibres. The lateral resistance induced by the fibres cannot be measured directly in peat. However, if the results

of triaxial tests are plotted in a Mohr diagram, the fibre resistance can be deduced if the shear strength

without fibres is known. Such a strength envelope may be obtained from ring shear tests, where the effect

Figures 25. Lateral reinforcing effects of fibres; apparent and actual stress conditions. (a) Apparent and (b) actual

forces on element along shear plane inclined at $45^\circ + \phi/2$ to horizontal (side length along shear plane = b). (c) Mohr

stress diagram and (d) corresponding force diagram.

of the fibres is almost negligible. This is particularly so after consolidation, because the fibres tend to

align themselves at right angles to the direction of the applied (vertical) stress; that is, in the direction of

the applied (horizontal) shear stress (Landva and Pheeney 1980, Landva 1980a). The strength envelope

in Figure 25 represents the general case with a cohesion intercept. The applied lateral stress is assumed

to be zero. At failure, if failure can indeed be reached, the apparent shear strength would appear to be

s_a , if the lateral stress is assumed to be zero. However, a lateral stress of σ_f would be required to bring

the stress circle at failure to touch the actual strength envelope. This lateral stress is roughly equivalent

to the internal resistance induced by the fibres.

3.2 Permeability (hydraulic conductivity) and hydraulic fracturing

The porosity of natural peat is of the same order of magnitude as that of loose sand, and the peat particles

are of the same general size and distribution as a silty sand and gravel. The permeability of the peat could

therefore be expected to be anywhere in the range 10^{-5} to 10^{-2} cm/s (Terzaghi & Peck 1967, Table 11.1).

Typical measured values of the permeability in unloaded peat are generally within that range. However, the permeability of natural peat is actually of little interest to the geotechnical engineer.

Generally, it is the permeability in a compressed or consolidated state that is of interest. Since the

voids through which water is permeating are getting smaller with applied load (elastic compression)

and with time (creep), the permeability must be expected to

decrease very considerably with load and

time. Measurements by Hanrahan (1954) and Adams (1965), replotted in Figure 26, indicate that the

relationship between the compression (or void ratio) and the logarithm of the coefficient of permeability

k is not always linear, as assumed by, for example, Miyakawa (1960), and Berry & Poskitt (1972).

According to Taylor (1948), k is a function of the average grain size (D_s), the density (ρ_f) and viscosity

(μ_f) of the permeating fluid, the void ratio (e) and the shape of the grains (C = shape factor):

Figures 26. Results of permeability measurements in peat as related to uniaxial compression.

Considering only the void ratio, and inserting the values of e given by Hanrahan (1954) and indicated in

Figure 26, the following values are obtained: Load, kPa
Load duration e k Measured k , cm/s
 $0 - 12.0$ $133D$ $2 s \times \gamma_f$
 $\mu_f \times C$ 4×10^{-4} 55 2 days 6.75 $40D$ $2 s \times \gamma_f$ $\mu_f \times C$ $2 \times$
 10^{-6} 55 7 months 4.5 $17D$ $2 s \times \gamma_f$ $\mu_f \times C$ 8×10^{-9} γ_f
 $/\mu_f$ will remain constant, D_s will decrease somewhat and C may be somewhat dependent on the void

ratio. However, the spectacular measured decrease in k , particularly that under the same load from 2 days

to 7 months, could only be accounted for by Eq. 1 if the void ratio were to decrease first to 1.1 ($n = 0.52$)

and during the 7 months to 0.15 ($n = 0.13$). Even if only the interparticle voids are considered, this seems

impossible under a load of only 55 kPa. Clogging of the voids also seems an unlikely explanation. Errors

in measurement are also unlikely; Briaud (1974) reports a k -value of 2.2×10^{-5} cm/s for an SH 5 peat

under no load, 2.6×10^{-7} cm/s under 35 kPa (1 day) and practically zero permeability (no measurable

flow through the 500 cm 2×20 cm samples) under 70 kPa (2 days). It would appear therefore that the

very low values of k under load are realistic. It may be hypothesized that the highly flexible yet open

structured peat particles are forced together under load and with time in such a manner that continuous

channels (voids) are virtually closed, at least locally. If a sufficient number of continuous voids are

closed, the permeability will become very low, while the porosity will remain high. Bjerrum et al. (1972) introduced the concept of hydraulic fracturing in clay and showed that such

fracturing can be due to horizontal as well as vertical cracking. Horizontal cracking in clay may occur

when the excess pore pressure u becomes equal to the initial effective overburden pressure $p' > 0$, i.e.

when the effective stress becomes equal to zero (Bjerrum et al. 1972). Any bond strength is thus not

taken into account. In peat, horizontal cracking would occur when:

where σ_v = tensile strength in the vertical direction. Since peat generally comes apart readily in the

vertical direction (separating along horizontal planes), σ_v is relatively low. Therefore, since $p' > 0$ in peat

is generally very low, perhaps zero, horizontal fracturing is very likely to take place at very low excess

pore pressures. One typical case would be a road embankment, where the excess pore pressures under

the fill could be expected to cause extensive horizontal fracturing in the adjacent unloaded peat.

3.3 Compression and consolidation features

Briaud (1974) found that rapid compression and rebound of SH 3-5 peat resulted in a practically complete

and relatively rapid rebound, while the application of a load over longer periods of time resulted in

progressively slower rebound, as well as less total rebound. These findings show that the compression

of the peat particles is time dependent. For very short periods of loading the behaviour of the peat

is essentially elastic. For progressively longer periods, creep or plastic deformation occurs and only a

certain part of the total strain energy is recovered by unloading. However, this recovery is not entirely

of an elastic nature; it continues after dissipation of (negative) pore pressures. It has been suggested (Adams 1965, Berry & Poskitt 1972) that the consolidation process in "fibrous

peat" involves the expulsion of the interparticle void water as primary consolidation, followed by the

expulsion of the intraparticle void water as secondary consolidation. However, as pointed out in article

1.4.4, the expulsion of interparticle and intraparticle water takes place simultaneously. Even in peat with

a low degree of humification, the openings in the organic membranes are ample for free drainage. This

is demonstrated visually in Figure 27 through a comparison of filter paper with SH 1-2 peat. At higher

degrees of humification, normally found in peat, part or all of the membrane has decomposed, leaving

a structure considerably more open to flow than that of the filter paper.

3.4 Strength considerations

The strength of peat in compression will be significantly affected by the fibre content, as outlined in

article 3.1. Thus the actual shear stresses under compression are much lower than the apparent shear

stresses, since the reinforcing effect of the fibres represents a lateral (internal) stress. This will be

discussed in more detail later. In simple shear the effect

of the fibres would be small or insignificant, since this mode of deformation

provides only limited fibre anchorage; nor are the flexible fibres capable of transmitting shear forces. In direct shear, vane shear, plate shear and cone penetration shear, excessive local compression

produces a fibre rope effect, and the peat thus acquires an artificially high strength unrelated to field

modes of deformation (Landva 1980b). This will also be discussed in more detail later.

3.5 Seepage effects

The effective stress in peat, as in mineral soils, may be expressed as:

where z = length of path of flow, γ' = submerged unit weight of peat, γ_w = unit weight of water, i = hydraulic gradient, h = hydraulic head. Since γ' of peat is generally zero or negative, it follows that only a very small hydraulic head is

generally required to make the effective stress zero (or locally negative). Since peat is very compressible, seepage forces may cause considerable compression. For example,

a hydraulic head of only 1.5 m, which would increase the effective stress by 15 kPa, could cause a

compression of about 10%. Seepage forces are therefore of importance in permeability testing.

4 LABORATORY INVESTIGATIONS

4.1 Index tests

There are divergent opinions on the definition of peats and organic soils, depending on the needs of each

professional group. However, the various systems have one feature in common: they are all based on the

content of organic matter. The content of organic matter is given in terms of ash content (A_c percent),

on the assumption that the percentage organic content is equal to $100 - A_c$. This assumption is based

Figures 27. Comparison between filter paper and sphagnum peat leaves. (a) Whatman filter paper (65X). (b) Ditto

(330X). (c) Ditto (600X). (d) SH1 leaves (65X). (e) SH 3 leaf (330X). (f) SH 1 leaf (1600X).

on the general conception that organic matter is "generally combustible matter, whereas the mineral

constituent, whether part of the plant growth or extraneous matter, is incombustible and ash-forming". One widely used method of determining the ash content is to fire an oven-dried sample in a muf

file furnace until the soil has been reduced to an ash. The temperature and the length of firing vary.

A temperature of 440 ° C and holding until completely ashed are the recommendations of ASTM Test

for Moisture, Ash, and Organic Matter of Peat Materials (D2974). The Muskeg Engineering Handbook

recommends 800 to 900 ° C for 3 h or "until the soil has obviously been reduced to an ash". Arman (1969)

recommends 440 ° C for 5 h, claiming that this temperature is very critical: a few degrees below will result

in incomplete combustion, and a few degrees above will result in a transformation of clay minerals and

hence a loss of mineral matter. MacFarlane and Allen (1964) compared the loss-on-ignition method and the dichromate-oxidation

method of determining organic content of a fibrous peat. They used a furnace temperature of about

800 ° C for the loss-on-ignition method. For the dichromate-oxidation method, which yields the content

of organic carbon only, they assumed, in accordance with normal practice, that the total organic matter

contain 58% organic carbon. On this basis, they tentatively concluded that most of the difference between

the two methods was due to water which had not been driven

off during oven-drying. In regard to oven

drying, it was found that charring (oxidation) of peat became increasingly evident at temperatures above

85 ° C. It follows that any attempt to drive off the rest of the water by increasing the oven temperature to

105 ° C or higher would result in a loss of organic matter. On the basis of these various investigations, the present author proposes that the water content of

organic soils be determined by oven-drying at a temperature not exceeding about 90 ° C and that the ash

content be determined by igniting at a temperature of 440 ° C for about 5 h. In cases of doubt, the ignition

method could be supplemented by other, more complicated (and hence more expensive) methods, such

as the dichromate-oxidation method. Reference is made to Jarrett (1983) with respect to tests for specific gravity, void ratio and unit weight.

In case of specific gravity, this can be estimated from:

where A_c = ash content and p_c = unit weight of constituents. This equation differs from Eqn. (4) in

the Muskeg Engineering Handbook (MEH) (1969, p.89). Indications are that the MEH equation is not

entirely accurate.

The term "ash content" is somewhat misleading, considering that there is no ash in peat, but rather a

mineral content that remains after firing the peat in a muffle furnace. The correct term would appear to

be "mineral content".

4.2 Consolidation tests (constrained deformation)

Conventionally, consolidation is regarded as the dominant process in the constrained deformation of a

compressible material, secondary compression being regarded as a "rather tiresome appendage" (Hobbs

1986). Both theoretically and in practice however, certainly in the context of peat, the roles are reversed.

The so-called secondary compression is “the dominant process, consistent and enduring, but temporarily

distorted by the hydrodynamic time-lag associated with [primary] consolidation, a mere aberration at

the outset of the stately march through logarithmic time” (ibid). That this must be so can be seen by

projecting the “secondary tail” obtained from an extended test backwards to the log time abscissa. The

masked secondary compression is assumed to follow the path AB shown in Figure 28. If the peat were

infinitely permeable, so that there was no hydrodynamic resistance, the compression would follow this

path in the laboratory and field. Hydrodynamic resistance, as the pore pressure dissipates, merely distorts

the path for [some] time into the familiar consolidation curve, the effect being greater in scale and longer

in time in the field than in the laboratory simply because the peat is thicker.

Where primary field settlements have been predicted on the assumption that the field primary strain

is equal to the laboratory primary strain under comparable pressures, observations in the field have

shown such predictions to underestimate the field settlement. For example, Samson & La Rochelle

(1972) found reasonable agreement, as others have done, when using the 24 hr readings from standard

multi-increment tests as the primary times, as is customarily done with works on clay. This is due to the

fact that in the standard 24 hr multi-increment test the secondary strains under each increment of loading

are automatically included in the successive primary

strains. The problem with this is to accurately

estimate the field primary time so that the appropriate amount of secondary strain accruing between the

primary time in the laboratory test and the estimated primary time in the field can be included. Weber

(1969), following his painstaking work on three San Francisco embankments, remarked that "attempts to

correlate between the rates of settlement indicated by the laboratory time settlement data and measured

Figures 28. Strain vs log time for two different thicknesses of peat under the same load (Hobbs 1986).

Figures 29. Consolidation tests on undisturbed peat from Escuminac. Tests P-67 (multi-step) and P-66 (single step).

settlement were unsuccessful". The virtual certainty of error or discrepancy between prediction and

performance, in the absence of large-scale field trials carried out in the same manner as proposed for

the works, is now generally recognized. In view of the dominant role of the secondary compression, time has to be introduced into the ϵ -log p

relationship. If, in a series of single increment tests on equal thicknesses of the same peat, a plot is

made of strain against the log of the pressure using the strains at the end of the primary stage and at

selected intervals of time thereafter, conveniently at log cycles, an ϵ - log p - t graph will be obtained as

illustrated in Figures 29 and 30. This is essentially the type of relationship discussed by Bjerrum (1967)

in connection with normally consolidated clays. The lines tend to flatten out at higher pressures, but

Figures 30. Consolidation tests on remoulded (P-82, multi-step) and undisturbed (P-66, single step) peat from

Escuminac.

within the loading ranges normally imposed by low embankments they are sensibly straight. A uniform

vertical spacing of the isochrones indicates a constant value of C_{α} sec with time, and parallel ones indicate

that C_{α} sec does not vary with pressure ($C_{\alpha} \text{ sec} = \frac{C_{\alpha}}{\alpha} \log t$).

The three paragraphs below, together with Figure 28, are excerpted from Hobbs (1986), a major paper

giving an unusually thorough and detailed account of consolidation in peat. Figures 29 and 30 show some typical results of laboratory consolidation tests on Escuminac peat. Tests

P-66 and P-67 represent undisturbed peat, and test P-82 represents peat reconsolidated from a slurry.

The most important findings from these tests, typical of a large number of tests, were (1) multi-step

(P-67) and single-step (P-66) loading led to the same amount of compression, (2) within the range of

pressures shown there was no significant difference in compression between undisturbed and completely

remoulded peat, and (3) there was no definite pattern in the rate of consolidation for the various tests

and load steps. It is a common observation that the coefficient of secondary compression ($C_{\alpha} \text{ sec} = \frac{C_{\alpha}}{\alpha} \log t$) as measured

under fills is generally considerably greater than that determined under corresponding pressures

from tests on ostensibly the same peat in the laboratory. For example, Lea & Brawner (1963) reported

ratios of field coefficient to laboratory coefficient of 0.83, 1.5, 2.5, 2.3, 4.1 and 4.6 from the highway

site in British Columbia, the average value being about 2.6. Lefebvre et al. (1984) obtained ratios of

at least 2 in Quebec. While the coefficients used by these authors are based on the thicknesses after

primary consolidation, this should in theory not affect the ratio, all other things being equal. Weber's

(1969) results based on the original thickness of the peat also confirm the observation. Various reasons

have been put forward for this very large discrepancy: variability of the peat in the field, not reproduced

by the small number of samples tested; loading conditions not strictly comparable; shear creep as a result

of the field conditions not being strictly one-dimensional; fluctuation of the water table; and heavy snow

increasing the load. These are all valid reasons, but two more may be added to the list:

1. The lumping together and averaging of laboratory results in depth without regard to morphology and stratigraphy; and
2. The tendency of C_{sec} to generally increase with time rather than to remain strictly constant. As the design of a pre-loading scheme, which normally requires projections to 30 years or so, is

heavily dependent on reliable predictions of the secondary compression and compression index, any

large-scale field trials should be maintained for as long as possible. This should deal with all the above

factors except the last, since normally it will not be possible to maintain the trial fill long enough to

investigate the variation of C_{sec} with time. It may be necessary to make some additional allowance in the

value of C_{sec} determined from the trial fill for this reason. A study of the laboratory consolidation behaviour of the Escuminac peat and a literature review of the

laboratory behaviour of other peats have not revealed a consistent picture of the consolidation behaviour

of peat. Even in cases where the peat appears to be reasonably uniform, e.g. at Escuminac, there are very

large variations in test results. It appears therefore that a settlement analysis on the basis of laboratory

tests at best may give only a rough estimate of the settlement to be expected. The initial permeability of peat is generally relatively high, and since peat also generally contains

a fairly significant amount of gas, there is always a substantial immediate settlement on application

of a load. The pore pressures at the end of this period are generally lower than the applied load. The

consolidation behaviour after the loading period depends on the rate of creep of the peat structure relative

to the rate at which water can drain out of the peat. Since the rate of creep of peat can be very high, it

would be possible in peats with a relatively low permeability (after the loading period) to have a period

of constant pore pressure. When the rate of creep becomes smaller than the rate of drainage, the pore

pressure will decrease, but the overall rate of consolidation is still lower than that in a structure where

there is no plastic creep. It should be noted that pore pressure dissipation could still be relatively rapid,

given high rates of both creep and drainage, and that the rate of consolidation (creep) would then continue

without any noticeable discontinuity in a settlement - log time plot. Quoting again from Hobbs' (1986) conclusions (next two paragraphs), "peat has a high coefficient

of secondary compression which, in terms of strain, is virtually independent of water content. This

so-called secondary strain is properly regarded as the dominant process which is merely distorted by a

short consolidation phase. This reversal of the traditional view of the consolidation process is essential

to the understanding, analysis and presentation of the results of tests under constrained deformation. It

is shown that the traditional notion of preconsolidation

pressure as an identifiable historically conferred

measurable parameter has no validity when applied to peat. The discontinuity observed in a void ratio

vs. log pressure plot is more exactly a "rheological parameter" (Leroueil & Marques 1996). It is a common experience that the performance of fills over mires cannot be accurately predicted by

laboratory testing. This misfortune, if such it be, is due not only to inappropriate methods of testing and

analysis but also to the higher permeability of fibrous peats in the field than in the small test specimens

used in the laboratory. Large scale trial fills are essential in important works". For preliminary and very approximate estimates, the compression index $C_c = \frac{de}{d \log \sigma'}$ may be

taken to be in the range $C_c = 0.0065w$ (Hobbs 1986, his Fig. 36) to 0.0115w (Azzouz et al. 1976).

However, settlement estimates based on a logarithmic relationship involving the original effective over

burden pressure σ'_o in the denominator would in general be inaccurate and could also be indeterminate,

since σ'_o could be zero or even negative. The settlement should be calculated simply from the relationship

where e_v is taken directly from a plot of e_v versus applied pressure or from a plot of the void ratio versus

applied pressure.

4.3 Stress-strain-strength behaviour of peat

There has not been an extensive amount of work on shear testing of peat. Hanrahan (1954) seems to have

been the first researcher to report such work. On the basis of undrained triaxial test results he concluded

that the strength of peat was exclusively cohesive in character. Adams (1961, 1965) carried out a series

of drained and undrained triaxial tests on normally

consolidated and over-consolidated undisturbed peat

samples of relatively low water content (200 to 600%). He concluded that the behaviour of the peat was

exclusively frictional in character, having a friction angle of 48° . He also measured a K_o value as low as

0.18, and he found that preconsolidation and anisotropic consolidation had little effect on the strength

parameters of the peat in triaxial compression. Gautschi (1965) carried out triaxial tests on peats of different degrees of humification. He drew

attention to the reinforcing effect of the fibres and pointed out that this effect must have been the reason

for the low K_o - values of 0.18 measured by Adams (1961). Hanrahan et al. (1967) carried out a comprehensive shear test program on remoulded peat. It is

interesting to note that the conclusions in this paper are quite the opposite of those in Hanrahan's

previous paper (1954), in which it was held that the structure of remoulded peat was unlikely to be

representative of undisturbed peat and that the shear strength of peat is cohesive only. In their 1967

paper, Hanrahan et al. stated that many previous tests had confirmed that the qualitative behaviour of

peat in its disturbed (remoulded) and undisturbed states is similar, and they concluded that the shear

strength of peat is essentially frictional ($c = 5.5$ to 6.1 kPa, $\phi = 36.6^\circ$ to 43.5°). Hollingshead and Raymond (1972) presented results of consolidated undrained and drained triaxial

tests. The undrained behaviour was erratic (5 tests) and no shear strength parameters were given. The

drained tests were terminated at 24% vertical strain without a peak strength having been reached and the

shear stress parameters for this strain corresponded to $c = 4$ kPa, $\phi = 34^\circ$.

The lateral resistance induced by the fibres cannot be measured directly in peat. However, if the results

of triaxial tests are plotted in a Mohr diagram, the fibre resistance can be deduced if the shear strength

without fibres is known. Such a strength envelope may be obtained from ring shear tests, where the effect

of the fibres is almost negligible. This is particularly so after consolidation, because the fibres tend to

align themselves at right angles to the direction of the applied (vertical) stress, that is, in the direction of

the applied (horizontal) shear stress (Landva and Pheeney 1980, Landva 1980a). The strength envelope

in Figure 25 represents the general case with a cohesion intercept. The applied lateral stress is assumed

to be zero. At failure, if failure can indeed be reached, the apparent shear strength would appear to be

s_a , if the lateral stress is assumed to be zero. However, a lateral stress of σ_f would be required to bring

the stress circle at failure to touch the actual strength envelope. This lateral stress is roughly equivalent

to the internal resistance induced by the fibres. Figure 31 is a summary plot of over twenty 240 mm diameter rings shear tests carried out on the

sphagnum peat from Escuminac. No individual tests results are included in this paper, but the following

conclusions from these tests may be drawn. The higher portion of the range shown in Figure 31 invariably

applies to tests where the settlement (δv) during shear was small or even zero. The strength for normally

consolidated peat sheared first time is represented by the shaded portion of the range. At normal stresses

above about 13 kPa this range is bounded by $c = 2.4$ kPa, $\phi = 27.1^\circ$ and $c = 2.4$ kPa, $\phi = 32.5^\circ$. At normal

stresses below 13 kPa the apparent cohesion increases and ϕ decreases, corresponding to a strength at

zero normal stress of about 5 to 6 kPa. There can be little doubt that this apparent cohesion is a result

of fibre entanglement rather than any type of cohesive bonds between the fibres. At low stresses such

entanglement would be quite prominent, while at higher stresses the flattening and reorientation of the

particles would tend to reduce the effect of the entanglement.

A common feature of the test results above the shaded range was that the settlement during shear

was low or even zero. This is attributed to a parallel, horizontal reorientation of the particles and a

densification of the peat structure as a result of (1) overconsolidation, (2) consolidation and shear

combined, or (3) shear under low normal stress followed by consolidation or reconsolidation at higher

normal stress. All test results above the shaded range fit into one or more of these categories. The ring

shear behaviour of fine-fibrous peat is therefore similar to that of loose sand (or perhaps loose sand with

a small content of short fibres). Only one test was carried out on highly decomposed (SH 8) peat. The behaviour and strength of this

peat in ring shear were essentially the same as that of SH 3-4 peat.

Tests on (1) remoulded peat, (2) undisturbed peat sheared at right angles to the in situ bedding planes,

and (3) undisturbed peat with a precut shear plane all gave results within the same range (shaded range,

Fig. 31) as normally consolidated peat sheared first time. This suggests that disturbance or remoulding

of peat samples is not critical and that it is the stress

and deformation history of the sample that is

important. This conclusion is also valid for all the other tests (above the shaded range), since it is

precisely the stress (overconsolidation) and deformation (shear) history of these samples that resulted in

their higher strength. Further examination of Figure 31 shows that the cyclic loading of sample 17 was within and partly

above the failure range, yet no failure occurred during the 85,000 cycles applied. The sample was clearly

stressed to its peak strength, but there was no discernible collapse or decrease of strength. After the

Figures 31. Ring shear test results for all tests (τ vs. σ_N).

cyclic loading, the sample behaved as if it were overconsolidated; that is, the fabric had become denser

and more orientated. The stress paths of the consolidated constant-volume (CCV) test extend into the upper range in

Figure 31. The reduction in vertical stress (37% during the first cycle, 15% during the second) corresponds

in effect to overconsolidation. This reduction in vertical stress to keep the height (volume) of the sample

constant is considered to represent an excess pore pressure, namely that which would be induced if the

sample had been sheared under truly undrained conditions (Taylor 1952, Bjerrum and Landva 1966).

The value of 37% is only slightly higher than that obtained by Landva (1962) for Norwegian quick clay

(33%), which has a readily collapsible structure. A series of consolidated undrained triaxial tests carried out by Hanrahan (1954) are plotted in Figure 32.

The induced pore pressure for all tests was approximately equal to the cell pressure at failure; that is,

the applied lateral effective stress at failure was zero.

If a strength envelope such as $c = 3 \text{ kPa}$, $\phi = 32^\circ$

(Fig. 31) is used in this diagram and the strength circles are replotted as in Figure 25, the resulting

lateral effective stresses at failure are representative of the lateral resistance induced by the fibres. This

resistance corresponds here to about $\sigma' \tan 16^\circ$; that is, a mobilization of an average friction angle of about

16° on horizontal planes. Other series of triaxial tests at lower stresses are plotted in the same manner

in Figure 33. The average fibre resistance for the A-series tests corresponds to $\sigma' \tan 11^\circ$, a relatively

low mobilization of fibre resistance, on account of a high pore pressure. The fibre resistance for the H 1

test corresponds to $\sigma' \tan 10^\circ$; while that in the less fibrous H 5 peat is $\sigma' \tan 3^\circ$ and that in the practically

non-fibrous H 9 is close to zero. The influence of the fibres is thus quite consistent. The mode of deformation of peat under relatively narrow embankments may be expected to be some

what similar to that in triaxial or biaxial shear; that is, the mobilization of lateral fibre resistance would

tend to restrict lateral bulging of the peat under the embankment under conditions of low pore pressures

(slow loading) and in peats with a high fibre content. Lateral bulging must, however, be expected to

occur under conditions of high pore pressures (rapid loading) and in peats with a low fibre content.

Figures 32. Triaxial tests on peat as plotted by Hanrahan (1954) (broken-line circles) and as corrected (full-line circles) for fibre reinforcement.

Figures 33. Triaxial tests on peat as plotted by Hanrahan (1954) and Gautschi (1965) (broken-line circles) and as corrected (full-line circles) for fibre reinforcement.

The shear strength of soils is usually considered in connection with shear failure, that is, complete

failure along one or several sliding surfaces or zones, associated with a definite and often considerable

displacement of the soil above the failure zone(s). Such failures were not observed directly for any of the

test fills at Escuminac, which varied in thickness between 0.6 and 7.0 m. The 7-m fill settled about 6.4 m;

Figures 34. Tension testing of peat. (a) Split tension box. (b) Mohr stress circles.

that is, the 7.4 m peat was compressed about 86%. About two thirds of this is judged to have occurred

under laterally confined conditions; the remaining 30% (2.2 m) would therefore be settlement as a result

of shear deformations. The shear settlement is thus very high and cannot be disregarded. However, a

knowledge of the shear strength of the peat would not be of any help in assessing the shear deformations.

Indeed, the only type of test carried out on the Escuminac peat that brought about failure, and thus

yielded shear strength parameters, was the ring shear test, and this test is not in any way representative

of the mode of shear deformations under an embankment. An assessment of the shear deformation to be expected may be made on the basis of large-diameter

(100 mm or larger) triaxial samples compressed vertically under unconfined conditions or under condi

tions of controlled lateral restraint. However, the field variables involved (the rate of loading, the partial

drainage conditions, and the partial lateral restraint) are of course unknown at the time of testing. A

large number of tests would therefore probably be required, and this would be a very time-consuming

procedure. Some idea of the shear deformation can be

obtained from a combination of confined consolidation

tests and quick and slow unconfined compression tests. The quick tests would represent rapid field

loading under partially drained conditions. The slow tests could be sufficiently slow to represent fully

drained conditions. In any case, the shear deformation under completely unconfined conditions would

represent a maximum, since in the field some lateral restraint does exist, as borne out by the passive

pressure test at Escuminac. The shear strength characteristics of Radforth peat as obtained from ring shear tests (Fig. 31) may be

considered representative of the conditions imposed by a water-retaining structure on peat with respect

to horizontal sliding. For example, the shear strength parameters for the Escuminac peat would be

$c = 2.4 \text{ kPa}$, $\phi = 27.1 \text{ to } 32.5^\circ$. For this mode of shear deformation, the triaxial test would not be suit

able and would in general give results on the unsafe side, increasingly so with increasing fibre content

(Figs. 32 and 33).

4.3.1 Tension behaviour of peat

The mode of failure in connection with the application of both compression and tension was first

identified as a shear mode when the writer was in the process of analyzing the results of laboratory

"tension" tests on fine-fibrous peat. As shown in Figure 34(a), the tensile force T was applied to a peat

sample consolidated under a vertical stress σ_{vc} (circle 2c, Figure 34b) and a horizontal stress $\sigma_{he} = K_o \sigma_{vc}$.

The value of K_o for this peat had been determined to be 0.20. The peat sample was anchored with brass pins through the plexiglass walls and failure occurred in a

zigzag pattern in the zone between a and b, as indicated in Figure 34(a). This zone would in fact be the

only location where failure could possibly occur, i.e. zone ab constituted a forced failure zone, similar to

the forced horizontal failure zone in a direct shear test. The tensile stress $-\sigma_t = -T_f / A_v$ (where T_f = the

applied tensile force at failure and A_v = the vertical cross-sectional area of the sample) had first - and

mistakenly - been considered to represent the fibre strength of the compressed peat. However, accounting

for the minor principal stress $\sigma_3 = -\sigma_t$ only and disregarding the major principal stress $\sigma_1 = \sigma_{vc}$ would

correspond to circle 1 in Figure 34(b), whereas the applied stresses clearly corresponded to the much

larger circle 2f. On the other hand, the same peat when brought to failure in ring shear had yielded

cd as the failure envelope ($c = 3 \text{ kPa}$, $\phi = 30^\circ$), representing the shear strength mobilized by sliding

along the more or less horizontal fibres (circle 2'). Failure envelope cd would thus correspond to the

shear strength of the peat without fibre reinforcement, since the fibres in this mode of failure would

not be stressed in tension. The corresponding zero-fibre-strength lateral stress at rest (circle 3) would

be $\sigma_g = K' \sigma_{vc} \approx 0.5 \sigma_{vc}$ where $K' \approx (1 - \sin \phi_m) / (1 + \sin \phi_m)$ and ϕ_m = mobilized angle of friction $\approx 20^\circ$

(Landva 2006). The contribution of the fibres to the lateral stress during consolidation was therefore

eg. However, the application of the tensile force T_f further stressed the fibres from e to $-\sigma_t$. Hence the

real fibre strength would be represented by $\sigma_f = |-\sigma_t| + \sigma_g$ (Vidal 1969, Landva & La Rochelle 1983).

In Figure 34, this would correspond to a mobilized friction angle of $\tan^{-1} 58/74 = 38^\circ$ (see article 3.1

above), suggesting some interlocking and intertwining of fibres. Alternatively, the fibre strength can be

expressed in terms of the cohesion intercept, in this case $c = 40 \text{ kPa}$. Finally, on examining the Figure 34(a) zigzag failure pattern and the Figure 34(b) Mohr stress circles,

it is seen that the sample failed in shear along planes inclined at $\alpha = \pm(45^\circ + \phi/2)$ to the horizontal, i.e.

the mode of failure was shear, not tension.

5 FIELD INVESTIGATIONS

5.1 Survey

5.1.1 Office survey

Peatlands consist of relatively young organic soil deposits that make up complex and sometimes con

stantly changing landforms. At the onset of any construction project in peatland it is prudent therefore

to start a geotechnical investigation with an office study based on remote data such as aerial photos

and maps. Existing topographic maps typically have scales between 1:10,000 and 1:250,000 and aerial

photos are normally available at scales ranging from 1:5000 to 1:100,000. Peatlands are generally underutilized in most locations, and the reason for this is simply that problem

areas as a rule are the last to be developed. Only limited detailed survey data are therefore likely to be

available.

The use of remote data can provide much useful information (Korpijaakko & Woolnough 1977):

- a. Extent of peatlands
- b. Local and global topography
- c. Site access
- d. Areas of open water

- e. Natural hydraulic conditions such as drainage
- f. Man-made features
- g. Extent of open or treed areas
- h. Peatland genesis (geological history)

Another advantage of aerial photos is that a comparative study may be made of aeri

als taken at different times. Finally, some indication of the depths of the deposits may be obtained from these studies, but this

does require considerable expertise, and it is generally only possible to divide the deposits into three

groups, viz shallow, deep and uncertain. If the office study indicates that the project approach is sound, the next logical step is ground

verification studies in the field.

5.1.2 Field survey

One important reason for proceeding to field verification studies is that the office study, however thor

ough, can not disclose the exact nature of the soil under the surface. This can be done only through

sampling and probing, and it is especially important to disclose, in as much detail as possible, dis

continuities such as stump layers, water-filled cavities ("black holes", rimpis) of dy and gyttja. Such

discontinuities are of much importance for the design and construction of structures on peatland.

Access to peatlands is often difficult for conventional field equipment such as ordinary drill rigs,

etc., but lightweight effective hand samplers are available for preliminary work. Hand samplers such as

Figures 35. SP200 screw plate load tests in intact and in precut peat (δ vs. time).

the Hiller borer can be used to obtain disturbed samples over the entire peatland profile, and it is also

possible to obtain crude samples of the mineral subsoil with such a borer. The field reconnaissance may be carried out on foot, if necessary aided by snow shoes, snow fence

matting, rubber dinghy, etc. If the project can afford it, and if landing is possible, a helicopter may greatly

facilitate access problems and speed up the investigations. Much information and an impressive array of meaningful data can be generated from this preliminary

investigation: - depth profile of peatland and classification of soils - micro and macro stratification of soil layers (provided that the samples, although disturbed, retain the original stratification) - microscopy examination of samples - geotechnical index properties, such as water content, consistency limits, remoulded shear strength, dry strength, mineral content (or organic content), specific gravity, fibre content, and pH and other chemical properties. This preliminary investigation should be coupled closely with the office survey. In fact, this couple

may in some cases be sufficient, so that it may not be necessary to proceed to the third step (detailed

field investigation). After the preliminary investigations it should in any case be possible to undertake an engineering

design evaluation, i.e. to assess the feasibility and possible limitations of the planned engineering works.

Decisions would also be made at this time whether to continue with more detailed investigations or to

proceed to construction.

5.2 Field testing and sampling

5.2.1 Plate load tests

Figure 35 shows the results of a plate load test carried out in the Escuminac fibrous peat. This type of

test was carried out to compare the load-settlement behaviour in intact and pre-cut peat. The pre-cut peat

was cut to a depth of 46 cm around the plate circumference, as shown in Figure 35. The contribution

of the fibres was found to be very significant (Landva 1980a). It stands to reason that the contribution

from the fibres decreases with increasing size of the loaded area, and when this area is as large as that

under a road embankment, the fibre effect can be expected to be negligible. Plate load tests in intact peat

cannot therefore be representative of full-scale field conditions. Plate load tests were also carried out with half plates behind vertical glass for the purpose of studying

the mode of deformation of the peat (Fig. 37). The vertical strain was found to reach a maximum some

distance below the plate and to decrease to zero at a depth of about two diameters (a behaviour somewhat

analogous to that of footings on sand). Failure invariably took place in the vicinity of the plate, where it

was both visible and audible as the compressed fibres were torn. Observations further indicated that a zone of compressed peat under the plate remained there during

further penetration after failure was reached. At this stage the mode of deformation became somewhat

similar to that of cone penetration. In fact, the failure loads for the screw plates were of the same order

of magnitude as those for the cone.

The following conclusions were drawn on the basis of the plate load tests in fibrous peat (Landva

1980a).

1. A considerable and indeterminate portion of the load applied to a load plate is taken by the surrounding

fibres. The fibre action is localized around and below the edge of the plate and becomes therefore

less important with increasing diameter of the plate. In the case of obstructions in the peat, such as

stumps and roots, the effects of these are detrimental to small (and regular-size) plates, but are less

significant under a road embankment.

2. Failure in the plate load tests is progressive and there is considerable volume change. Local failure

starts at a very early stage around the plate edge. After a certain settlement has occurred, a maximum

value of applied stress may be reached, but it is difficult to define the corresponding mode of failure.

3. In summary, it may be stated that the results of plate load tests in peat are indeterminate with respect

to both compression and failure. The plate load test in peat is not representative of any real mode

of deformation of structures on peatland and is therefore of little geotechnical significance. One

exception does exist: it may be used as an indication of bearing capacity for concentrated loads on

the peat mat, such as vehicle traffic.

5.2.2 Static cone penetration tests

A series of static cone penetration tests with a 300-mm diameter cone was carried out in the Escuminac

peat. Figure 36 shows the results of one test for which the cone was equipped with a pore pressure filter

and transducer, located as shown in the figure. The induced pore pressures were negative throughout

the penetration. Below about 4 m depth the negative excess pore pressures corresponded to 20-30%

of the applied pressure. All tests carried out with the cone showed that an initial penetration of about

300 mm was required to mobilize the full resistance of the peat. The mode of deformation of the peat

was investigated separately with a half-cone behind vertical glass walls (Fig. 37). The results of these

tests have been reported by Landva (1980a).

The various cone penetration tests led to the following conclusions:

1. The large vertical compression required in cone penetration testing to mobilize the strength of the peat,

and the observed expulsion of water from the peat surrounding the cone, show that a considerable

and uncontrolled amount of vertical consolidation occurs prior to failure.

2. The horizontal displacement of the peat and the resulting tension in the mainly horizontal fibres also

leads a generation of negative pore pressures around the lower part of the cone.

3. It can be expected that a regular-size cone (say 50 to 75 mm diameter) will be readily deflected by

obstructions such as stumps and roots. Such obstructions are usually plentiful in fibrous peats.

4. The mode of deformation in fibrous peat is one of varying compression within a zone of about two

diameters and one of tearing within a zone of about 1.5 diameters. The layers that are punctured by

the cone point are displaced and compressed more and more as the cone continues its penetration.

Maximum compression and displacement occurs of course at the edge. The cone penetration failure

pattern does not resemble any real mode of deformation under structures on peatland, nor does it lend

itself to a rational interpretation of either compressibility or strength. The cone test in peat is therefore

of little engineering use.

5.2.3 Vane test

A series of vane tests was carried out at Escuminac with vanes ranging in size between 25×50 mm

and 300×600 mm and for rates of rotation varying from 0.02 to $15^\circ/\text{sec}$ (Landva 1980a). These tests,

together with a survey of the literature on vane testing in peat led to the following conclusions:

1. Sphagnum moss peat within the von Post classification range of about

SH 2-6 B 3-5 F 0-1 R 0-1 (N) may be expected to fail in vane shear at true angles of rotation of about 20° - 40° .

The failure surface is cylindrical in shape. Its diameter is about 10 mm greater than that of the vane.

Figures 36. Results of cone penetration test CPU with pore pressure measurement (q_c and u vs. depth), 300 mm

diameter cone, test area II.

Figures 37. Comparison of modes of deformation behind plexiglass. 155 mm half plate and 126 mm diameter

half cone.

A void is formed behind each blade as the peat is brought to failure, and the peat in front of each blade

is compressed tangentially in a regressive manner. The maximum compression occurs immediately in

front of each blade, whereas the peat adjacent to the void behind the next blade is not compressed at

all. As the peat is compressed in the tangential direction, it expands in the radial and axial directions.

The resulting confining pressure is a function of the properties of the peat.

2. The vane shear strength measured in both virgin (unloaded) and loaded moss peat is for a large part

an artificial strength brought about by angular compression

and by radial expansion. This mode of

deformation and failure leads to an abnormally high local fibre strength of the same nature as that in

a fibre rope.

3. Vane shear testing of fibrous peats with standard size vanes results in the measurement of an artificial fibre strength only, and no distinct failure surface is formed. Such testing is therefore of little engineering use.

4. The friction between peat and uncased rods may be considerable and appears to be erratic and inconsistent.

5. The measurement of angles of rotation may be hampered by rod twisting and by yielding of threads. It seems difficult to produce a satisfactory calibration curve of torque versus twist, even under controlled laboratory conditions.

6. The method of inserting the vane into the peat has a bearing on the results obtained. A blunt-edge vane pushed into fibrous peat or even moss peat may act as a load plate, causing both failure and compression of the peat before vane shear. A sharp-edge vane tapped into moss peat with sharp blows (high impact) appears to cause little disturbance to the peat.

7. The rate of rotation of the vane had little or no effect on the apparent measured strength of the sphagnum peat. Failure of the peat appeared to occur under essentially drained conditions within the entire range tested ($0.02-15.0 \text{ } ^\circ/\text{s}$). Considering (i) that the observed mode of deformation of peat in vane shear bears little, if any,

resemblance to that under structures on peat, (ii) that the measured strength in both fibrous peat and

moss peat is for a large part an artificial fibre matting strength, (iii) that both stresses and strains are

variable and unknown, and (iv) that vane testing of peat to date has not led to any consistent design

criteria, it must be concluded that the vane test in peat is of little engineering use and that it can in fact

be directly misleading.

5.2.4 Sampling

Disturbed samples of peats and organic soils may be obtained with a Hiller sampler, a Davis sampler or

a Macaulay sampler. Undisturbed samples can generally be obtained with special piston samplers. A 100-mm diameter piston sampler developed at the University of New Brunswick has been used

successfully in fibrous peats as well as in extremely soft, almost liquid organic bay sediments. Special

equipment had to be developed for guided trimming and preparation of the samples for laboratory testing,

as described later and also as described by Landva et al. (1983). Large-diameter samplers that appear to be suitable for organic soils and probably also for most peats

are (i) the 250 mm diameter Sherbrooke sampler (Lefebvre & Poulin 1979, Lefebvre et al. 1984), (ii) the

200 mm diameter Laval sampler (La Rochelle et al. 1981), and (iii) the 150 mm diameter UNO sampler

(Rowe et al. 1984). The 100 mm UNB piston sampler has a Plexiglas tube insert to hold the sample. During sampling the

Plexiglas is protected with a bronze tubing. The cutting edge is of stainless steel, as is the remainder

of the 100-mm tubing. Undisturbed samples are obtained through the combined action of suction and

cutting. A double leather cup piston is used for the creation of maximum possible suction, and the

stainless steel edge is maintained at an angle of 30 degrees with the vertical. For sampling in extremely

soft soil of almost liquid consistency or in very loose organic silts, the sampler may be equipped with a

foil retainer adjacent to the cutting edge. This sampler has been used successfully to depth in excess of

7 m in sphagnum peats at Point Escuminac and to varying depths in other types of organic soils. The UNB piston sampler was designed specifically for the testing of 100-mm

diameter specimens.

It forms the basic unit for a complete system consisting of sampler, trimming equipment, and testing

apparatus. The latter comprises triaxial testing with lateral strain control, consolidation tests, and simple

direct shear tests. The success of the UNB sampler in peat, organic soils and harbour muds may be ascribed to the

combined action of suction and a sharp cutting edge. There is no significant loss of moisture on removing

the sample from the sampler, since it is not extruded from the Plexiglas tube in which it was sampled.

Also, the length of peat sampled is 60 cm, while only the middle 20 cm length (in Plexiglas) is considered

to be undisturbed. Swelling is insignificant so long as cutting and sealing are carried out rapidly. Landva et al. (1983) concluded that "the sampling of peat no longer presents any difficulty so long

as the sampling site is accessible with a tracked vehicle". This conclusion is certainly valid for the type

of peat found at Escuminac (Fig. 22) as well as for soft organic soils and harbour muds, but it proved

not to be valid for a peat in a blanket bog at St. Shotts, Newfoundland. Sampling was attempted at about

1 m depth in a 2 m deep layer consisting of a mixture of sedge-moss, woody-sedge-moss and sedge

peat, possibly containing some scirpus roots. The exact type of peat at the chosen sampling location is

somewhat uncertain, but it did contain a significant amount of carex. This type of peat proved to defy

Figures 38. Field permeability test, test arrangement.

any attempt to be sampled with the 100-mm sampler, despite the sharp cutting edge of the sampler and

attempts to cut the sedge fibres by tapping the sampler rather than pushing it.

5.2.5 Permeability (hydraulic conductivity) testing

As noted in article 3.2, the probability of hydraulic fracturing in peat is high since the vertical effective

stresses are very low. A typical value for σ'_{v0} is 3.7 kPa (Fig. 40b), hence hydraulic fracturing along

horizontal planes could be expected at heads as low as about 0.4 m. Permeability tests were carried out at Escuminac in pits excavated to below ground water level.

The tests were carried out at approximately constant head in a recharge mode (positive head). The

piezometers were of the GEONOR type (32 mm diameter sintered bronze filters). They were installed

at depths between 2.4 and 5.6 m depth below the top of the peat.

The lowest initial constant head applied was 0.05 m. It was increased in steps of 0.05 m up to about

0.3 m and then in increasingly higher steps up to a maximum head of over 6 m. A ladder and pulley

arrangement was used for the higher heads. Details of the test set-up are shown in Figure 38. The initial head for each test was h and the final

head $h + \Delta h$. When Δh was small relative to h , the tests were considered to be constant-head tests with a

head $h + \frac{1}{2}\Delta h$.

Figures 39. Relationship between coefficient of permeability and void ratio in peat as reported by various

investigators. On plotting the test results as the rate of flow (q) vs. the hydraulic head (h), a definite discontinuity

was observed at $h \approx 80-110$ cm, i.e. k increased more or less suddenly at this head. The vertical effective stress in the peat below 0.9 m depth is about 3.7 kPa (Fig. 40b). It is practically

constant with depth below 0.9 m, since the submerged unit

weight for a water content range of 1000 to

1600% and a degree of saturation of 90-95% is only 0.02 to 0.03 kN/m³. Horizontal fracturing would

occur if $u \geq \sigma'_{\theta} + \sigma_{vt}$ where u is the excess pore pressure, σ'_{θ} is the effective overburden stress and σ_{vt}

is the tensile strength of the peat in the vertical direction. Helenelund (1968) found the vertical tensile

strength of a Canadian sphagnum peat to be in the range 4-7 kPa. Thus horizontal fracturing could be

expected at $h = u/\gamma_w = [3.7 + (4 \text{ to } 7)]/9.81 = 0.78 \text{ to } 1.09 \text{ m}$, and this does in fact correspond to the

observed discontinuity. The permeability of natural (uncompressed) peat is, notwithstanding all possible care taken to avoid

fracturing or compression, not of great importance in foundation engineering. It does serve, if properly

measured, as an initial value, but this value changes immediately on application of a load and may become

several thousand times less with load and time (Figs. 26 and 39). Also, it will increase significantly on

hydraulic fracturing. A typical case is the decrease of k under an embankment as a result of compression,

and the likelihood of hydraulic fracturing and thus an increase in k in the adjacent unloaded peat.

Field measurements of k for the prediction of the peat behaviour under embankments are therefore not

applicable. If carried out in steps of increasing head from small values of h , field measurements would

indicate the occurrence of hydraulic fracturing, but they will not disclose the more important relationship

between permeability and compression, nor will they disclose any decrease in permeability with time.

These relationships could only be predicted by conducting

permeability tests on undisturbed, relatively

large samples in the laboratory. It is important that seepage pressures are taken into account in the laboratory, since the seepage

pressure exerted by the water represents an applied effective pressure of about 10 kPa for every one

metre of hydraulic head. This is of course very significant when dealing with a material as compressible

Figures 40. Passive pressure test at Escuminac. (a) Section with point of application of lateral force (required to keep

walls parallel). (b) Effective overburden pressure as calculated from weight of block samples. (c) Passive resistance

as a function of lateral displacement.

as peat. It follows that the seepage pressure must be taken into account in calculations of consolidation

pressure. It also follows that it would be practical to start the flow at the same time as the load is applied,

rather than starting it later and having to wait for further consolidation as a result of seepage.

5.2.6 Passive pressure test

A test pit was excavated by chainsaw and flat shovel at Escuminac in peat of the type shown in Figure 22.

The pit was 1.57 m deep and measured 1.32 m by 3.38 m in plan. Timber walls measuring 1.83 m

in length and 1.55 m in height were installed along both sides, protruding 0.20 m above bog level.

Figures 41. Passive pressure test in test pit at Escuminac. Body diagram and force diagram for Rankine wedge,

$\phi = 30^\circ$, $c = 2.4$ to 3.8 kPa.

The effective height of the walls was therefore 1.35 m. The walls were terminated 0.05 m above the

bottom of the pit. The load on the walls was applied with a hydraulic jack, at the depth shown in Figure

40. The point of load application was found by trial and error, starting by applying a load of 12 kN at

a depth of 0.90 m below bog level (Fig. 41). Subsequent adjustment led to the final position of 1.02 m

below bog level, as shown in Figure 40. The duration of the test was about 4 hours. It was assumed that this was sufficient time for excess

pore pressures to dissipate, i.e. it was assumed that drained conditions prevailed. A schematic cross section through the pit is shown in Figure 40a, and the effective overburden pressure

is given in Figure 40b. The latter is based on weights of large block samples cut during excavation of the pit. The loading conditions in the passive pressure test may be considered to have been close to a Rankine

state, considering that there was little friction between the wall and the peat (the weight of each wall

is 3.4 kN, corresponding to a shear stress of only 1.4 kPa). However, a calculation of Rankine passive

pressures on the basis of $c = 2.4$ kPa and $\phi = 30^\circ$ (average from ring shear tests, Fig. 31) gives a resultant

force P_p of only 38 kN as compared to the measured force of 52 kN. Also, the theoretical point of location

is only 0.77 m from the bog level, as compared to 1.02 m as applied. The calculated value of P_p is very sensitive to the value of c . Thus an increase of c from 2.4 kPa to

only 3.8 kPa will increase P_p from 38 to 52 kN, as measured. The reason for this sensitivity is of course

the very low effective stresses involved. It will be seen from the ring shear results (Fig. 31) that the value

of c at low effective stresses may approach values as high as 6 kPa.

Figures 42. Tracing of an aerial photo showing crack pattern. It is not possible to draw firm conclusions on the

basis of one test only. Indications from one single test are that the measured value of P_p is in the Rankine range, while the actual point of application is considerably lower than that corresponding to a Rankine state.

Average values of s and σ' along a passive Rankine failure zone may be calculated on the basis of a

rigid-body motion of the passive wedge. This is done in Figure 41. The average values of σ' and s for

$c = 2.4$ kPa and $c = 3.8$ kPa are plotted in Figure 31. As might be expected, the stress range in the passive

pressure test was very low.

5.3 Instability of Escuminac peat cliff

The Point Escuminac peat bog forms a peninsula bordering on the Miramichi Bay in New Brunswick. The

peat shoreline is about 6 km in length. Part of the geotechnical investigation program at Point Escuminac

was carried out at the peat cliff along the shoreline. During the course of selecting suitable test sites

there, it was discovered that several cracks exist in the peat at distances up to about 40 metres from

the cliff. The cracks generally form concentric semicircular patterns, often interconnected by cracks

running approximately parallel to the shoreline. The larger semicircles approach 100 metres in diameter

(Figs. 42 and 43).

The problem of peat cliff stability could probably not be claimed to be of particularly great importance.

However, the unusual but consistent failure pattern did present a challenge towards furnishing a rational

explanation of the geotechnical mechanism involved.

Three aspects of the peat cliff formation are of primary

geotechnical interest: (i) the cliff is almost everywhere vertical, (ii) the thickness of the peat decreases towards the cliff face, from about 7 m at the center of the peatland to about 5 m at the cliff face, and (iii) a definite pattern of semicircular, concentric cracks exists more or less regularly in the peat along the shoreline.

Triaxial tests on sedge peat (Hollingshead and Raymond 1972) and ring shear tests on sphagnum peat

(Fig. 31) have shown that peat behaves essentially as a frictional material, having friction values in the

range 27° to 34° and cohesion values of 3-4 kPa at low stresses. This range of c-values is much too low to

account for the fact that a 5 m unsupported vertical cut can exist in the peat. However, if the reinforcing

action of the peat fibres is taken into account, the existence of such cuts is readily understood. This

situation is analogous to that occurring when fibreglass matting or bales of hay are compressed at right

angles to the fibres: the discontinuous fibres are stressed in tension, and this stress is a function of the

applied stress, the angle of friction between the fibres, and the strength of the fibres. Also, the more

the material is compressed, the higher becomes the fibre strength per unit volume, since the number of

fibres per unit volume increases. It is obvious from the very existence of the peat cliff that the Point Escuminac peat formation originally

covered a larger area, now occupied by the shallow waters of the Miramichi Bay. Also, it seems clear (a) (b)

Figures 43. Aerial photos of peat cliff.

Figures 44. Schematic illustration of forces acting on peat between cracks near cliff.

that peat cannot be highly resistant to erosion by wave action, particularly not during heavy storms.

However, the existence of the concentric semicircular crack pattern in the peat behind the cliff can not

be explained directly by erosion and is in fact a geotechnical phenomenon directly attributable to the

engineering properties of the peat.

An examination of the crack pattern (Figs. 42 and 43) shows that the cracks extend either out to the

shoreline or they are interconnected with other cracks extending to the shoreline. It is this observation

that provided the key to an explanation of the existence of the cracks, as follows. In Figure 44, crack ce represents the innermost (open) crack. The various forces on the block are

indicated by arrows and referenced in Figure 44. In order for a crack to form along fh (behind the existing

crack ce) the peat would have to fail in tension along fh. Such a failure mechanism is conceivable if the

tensile strength of the peat is exceeded by a horizontal force acting in the direction of the cliff. Crack

ce is at least partly open and communicates with the shoreline. The water level in ce is therefore well

below that in the potential failure plane fh. It is shown below that a differential head of only about 1.2 m

is sufficient to cause tension failure at fh.

The dimensions corresponding to the inner block of Figure 44 are $ec = fh = 5.4$ m, $eh = cf = 2.8$ m,

and $hg = 0.3$ m. The strength of the mat is taken to be 28 kPa (Jarrett 1976, Lee and Jarrett 1978)

and that of the peat 5.5 kPa (Pheeney 1976). These values give forces of $T_m \approx 8$ kN/m and $T_p \approx 28$ kN/m.

The water force on fg is $U_R \approx 128$ kN/m. The shear forces S_R and S_L may be disregarded because of

the very low shear modulus of the peat in bending. The lateral effective force $P' L$ on ce can also be

disregarded, since ce , owing to the fibre action in the peat, is at least partly open. Assuming now that point d is 1.2 m below g , the water force on cd is $U L \approx 75 \text{ kN/m}$. Since

$W \approx 2.8 \times 5.1 \times 10 \approx 143 \text{ kN/m}$ ($\gamma_{\text{peat}} \approx 10 \text{ kN/m}^3$) and $U B \approx 124 \text{ kN/}$, the normal effective force on

cf is $Q' v \approx W - U B \approx 19 \text{ kN/m}$. The difference between $U R$ and $U L$ is 53 kN/m , which exceeds $T M + T p = 36 \text{ kN/m}$ by 17 kN/m .

The maximum value that $S B$, the shear resistance along the base, can have is approximately

$Q' v \cdot \tan N + 2.8 c$, and with the N and c values quoted above, $S B_{\text{max}} \approx 21 \text{ kN/m}$. Thus the total resistance

is $T M + T p + S B_{\text{max}} \approx 57 \text{ kN/m}$ while the total driving force is $U R - U L \approx 53 \text{ kN/m}$. If the head difference

is increased from 1.2 m to 1.3 m, the driving force would exceed the resistance. However, tensile failure

would probably be initiated even before the head difference reached 1.2 m, considering the extremely

low compression modulus of peat. It is concluded that the cracks forming in the peat in semicircular concentric patterns are tension

cracks which develop progressively as a result of seepage forces in the peat. The seepage forces are

brought about by local drainage in open cracks extending at both ends to the shoreline or to other cracks

draining to the shoreline. The behaviour and properties of peat deviate in many respects from those of inorganic soils. However,

this study of the instability of the Escuminac cliffs has indicated that peat behaviour can be explained

on the basis of ordinary geotechnical principles as long as the special properties of peat are recognized.

6 PRACTICAL APPLICATIONS, FIELD BEHAVIOUR

6.1 Embankments on peat over firm subsoil

6.1.1 Escuminac test fills

A series of nine test fills (TR1 to 9) of thickness between 0.6 and 6.9 m and widths between 3 and 10 m

were constructed at Escuminac at a location where the peat had a uniform thickness of 7.0 to 7.4 m and

was underlain by glacial till which had a practically level surface. Various stages of the deformation of the Escuminac peat under and adjacent to the fills were apparent

from the observed behaviour of test fills TR1 to TR9. Three stages could be identified:

1. Under a moderate amount of fill, say up to one metre, considerable settlement occurred at the centre. The settlement decreased toward the shoulders due partly to the lower stresses there and partly to the tensile strength of the mat (membrane effect).

2. At a fill thickness of 2 to 3 m, the bowl-shaped compression was much more distinct, and the mat was stretched almost to failure in tension.

3. At a fill thickness of more than 3 to 4 m the mat failed in tension and considerable water and gas inflow into the adjacent peat took place. The settlement histories of four of the test fills are shown in Figure 45 for a time period of about

20 years. The stress ranges on the curves represent the stresses as first applied (higher values), followed

by the stresses after settlement (lower because of increased submergence). The rates of settlement, expressed as

decreased with increasing stresses, but this cannot be considered as a generally valid relationship since

the time at the start of the C sec linearity varied erratically and over an extreme range (one to 500 days). The behaviour of test fill TR8 (max. applied pressure = 86 kPa) is of special interest since this fill

caused (i) complete failure of the mat and the upper portion of the peat, (ii) extensive bulging of the

peat below the fill, and (iii) extensive outflow of groundwater into the unloaded peat adjacent to the

embankment, accompanied by hydraulic fracturing. Figure 46 shows the construction history of TR8, and Figure 47 depicts sections through the fill up to

four years after construction. With reference to Figure 47c, the volume of water in the peat between the

centreline and line ab before construction was $40 \text{ m}^3/\text{m}$ if the average water content of the peat is taken

to be 1300%, $G_s = 1.5$ and $S_r = 0.95$. The corresponding volume of water in the rockfill at the end of

construction would be $10.3 \text{ m}^3/\text{m}$ if the porosity of the rockfill is taken to be 0.40, and the volume of

Figures 45. Settlement during and after construction of test fills at Escuminac.

Figures 46. TR8: settlement under the centerline and heave adjacent to the fill (maximum applied pressure = 86 kPa).

water in the compressed peat beneath the fill was about $19.4 \text{ m}^3/\text{m}$ on the basis of porosities varying

with the strain across the section, including zone bfi (lateral bulging). The net volume of water squeezed

out would thus be $40.0 - (10.3 + 19.4) = 10.3 \text{ m}^3/\text{m}$. In comparison, the observed heave next to TR8

(Fig. 47) corresponds to about $11 \text{ m}^3/\text{m}$. This good agreement is considered to confirm the conclusion

Figures 47. Sections through large test fill TR8 at end of construction (1975) and four years after construction

(1979). (a) Cross section at settlement plate C. (b) Longitudinal section on centerline. (c) Cross section at settlement

plate C. Note heave extending to adjacent test fill TR7.

that the heave (swelling) adjacent to the fill was caused by water inflow. Further confirmation of this is

the observation that practically all the swelling had subsided some four years after construction.

6.1.1.1 Conclusions on test fills

The behaviour of the test fills at Escuminac and a study of several other case records of similar fills have

led to the following conclusions pertaining to the behaviour of true peat under embankments on peat

over firm subsoil:

1. No sliding or spreading failures took place for any thickness of fill (0.6 to 7.0 m) regardless of the rate of placing. The measured maximum pore pressures were generally smaller than the applied loads.
2. The lateral displacement (i.e. the shear deformation) of the peat became increasingly significant at applied pressures greater than about 40 kPa. For narrow embankments such deformations result in increased settlement.
3. The heave of the peat adjacent to the fill is mostly gas and water inflow (horizontal fracturing). The shear deformation (zone bfi, Fig. 47) appears to be limited to a lateral compression of the peat without causing any heave of the surface.
4. The addition of berms has no stabilizing effect on the peat. The weight of the berms does, however, prevent or reduce heave of the adjacent peat. It also increases the settlement of the fill under the shoulders.
5. The pore pressures under the fills in most cases dissipate within 2-3 months after construction. The rate of settlement did not appear to be a function of the rate of dissipation of pore pressure.

Figures 48. Settlement (compression ϵ_v) under various embankments on peat.

6.1.2 Other case records

Figure 48 is a compilation of (i) all Escuminac laboratory consolidation tests, (ii) one typical consol

idation test on a fibrous peat (H_3 , $C_c = 8$, $e_0 = 16$; Hobbs 1986), and (iii) the settlements of different

embankments observed two weeks to 19 months after construction. The shaded zone is a combination

of these settlements, but not including settlement due to shear deformation (NBR-2 and Escuminac test

fills exceeding about 50 kPa applied pressure). In the case of the test fills, their settlements for applied

pressures below about 50 kPa are lower than the shaded zone range because of the mat resistance, which

was considerable. (A comparison may be made between the effects of the strong mat in the field and the

influence of the small but numerous fibres in peat below the mat, shown in Figures 35 and 49.)

The co-ordinates of the shaded zone have been replotted in Figure 50, giving a value of $C'c = 0.49$.

This value would apply for the case of no mat reinforcement, i.e. the mat is considered to have yielded

its resistance to embankment settlement (cf. 1967 road, Fig. 48).

Figures 49. 299 mm octagonal cell test R-76-1. Recorded deformation patterns at $\sigma_N = 36.2$ kPa and $\sigma_N = 51.1$ kPa

(1/3 of sample width loaded).

Figures 50. Shaded zone of figure 48 replotted to $\log \sigma_1$ scale.

The C_{sec} - values from Figure 45 give $C_{sec}/C'c$ values in the range 0.05-0.09 as indicated in Figure 50.

This range is in general agreement with values quoted by Mesri and Godlewski (1977, their table 2).

6.1.2.1 Some other conclusions

1. The total settlement of fills on peat varies within very wide limits within any one peat deposit even if

the applied load and the depth and type of peat are the

same.

2. The rate of settlement of fills on peat is largely unpredictable on the basis of laboratory or field testing,

including test fills. The generally accepted linear relationship between settlement and log time did

apply to some of the fills observed over a 1-2 year period, but was not borne out by the Escuminac test

fills (observed over periods of up to 15 years after construction) and not by some other case records

of observations over shorter periods.

3. The compression of peat may vary considerably with depth despite only small variations in water

content and degree of humification. This observation may be one reason for the discrepancy often

existing between settlement predictions and observed settlement.

6.2 Traffic load tests

A series of traffic load tests was conducted with a fully loaded tandem truck on the 0.7 m thick test fill TR5.

The truck was backed out on the test fill about 60 times in all. Electric piezometers had been installed,

and these indicated that $B = \Delta u / \Delta p$ was in the range 0.6-0.8, most of the values being around 0.6.

A review of the various reports on road failures on peat had suggested that traffic loads would cause

pore pressures to build up gradually in the peat, particularly around mid-depth, where drainage would

be slowest. However, the test fill truck load test series indicated that, although excess pore pressures are

indeed induced by traffic loads, they dissipate almost instantaneously, and little or no residual excess

pore pressures remain after the loads have passed.

The flexible underlay under the TR5 fill (5 cm compacted sand between Mirafi 140 fabrics) yielded

considerably under the concentrated wheel loads. Evidently the flexible underlay provided only a limited

load spreading capacity. One single load test with a fully loaded truck was carried out on the old (ca.1870) road at Escuminac.

The thickness of the well compacted fill was 0.4 m, and it was underlain by corduroy consisting

of a transverse layer of 5-12 cm diameter tree trunks (no spacing) and three longitudinal stringers.

Figures 51. Sections through main road at sta 33 + 50 m and 33 + 70 m.

The maximum compression of the peat was 3 cm during loading, and the peat rebounded completely

(within surveying accuracy) upon unloading. There was no discernible wave action. It was evident that

the stiffness of the corduroy, combined with the compact fill, provided a highly efficient load spreading

capacity. On the basis of visual observation, the performance of this 0.4 m thick road embankment was

at least as good as that of the 1.2 m thick test fill TR6. A series of traffic load tests was also conducted on the main (1967) road around chg. 33 + 50 m. Four

fully loaded double-axle trucks were driven back and forth on the road between approx. chg 32 + 50

and 34 + 50 for two consecutive days, mostly in double file (Fig. 51) configuration. Several piezometers

had been installed under the road, as shown in Figure 51. The readings in the electric piezometer P2

varied between 3 and 5 kPa, with one maximum reading of 7 kPa. A Boussinesq type stress distribution

gives a maximum additional stress at P2 of 7 kPa, which occurs under the centre of four rear axles, i.e.

under the centreline of the road with two trucks side by side. Several things of geotechnical interest were learned or confirmed from this series: (i) Pore pressures

corresponding to B-values of 0.4-0.7 (1.0 in one single case) developed instantaneously as the moving

load passed over the peat (as was the case with the truck load tests on test fill TR5). The pore pres

sures dissipated immediately when the load had passed. This conclusion is valid for any number of the

passes made, from the first pass on the first day to the last on the second test day. The total number

of passes was approximately 800 by each truck. (ii) The pore pressures remained constant or increased

slightly when the trucks were parked on the test section (the maximum increase was about 0.8 kPa dur

ing a one-hour parking). (iii) Pore pressures in peat measured with standpipe type piezometers are not

reliable on account of the gas content in the peat. It is difficult, if at all possible, to saturate the system.

(iv) Numerous hair-cracks developed in the road as a result of the truck loads. Most of the cracks were

in a transverse direction. No other signs of failure were apparent. The total (net) settlement of the road

as a result of the test series was 1 to 2 cm. The road embankment oscillated ± 0.5 cm when one truck

was passing and ± 1.0 cm when all four trucks were passing. (v) On excavating trenches across the road

after the test, the fill at one location (33 + 70 m, Fig. 51) was found to have a maximum thickness of

only 0.5 m. It was underlain by transverse trunks of maximum diameter 12 cm. The spacing between

the trunks was of the order of several centimetres. No longitudinal stringers were found. It is seen from

Figure 51 that the road embankment was as thin as 0.2 m

under the outer wheel loads (up to 60 kN), yet

the road suffered no damage other than the transverse hair-cracks. It may be speculated that even these

cracks may have been reduced, or may not even have formed, if stringers had been present and if there had

been no spacing between the trunks. It is noted that there was no discernible difference in performance

between chg. 33 + 50 and 33 + 70, despite the very considerable difference in embankment thickness.

A comparison of the shapes of the bog surface under the fill after the truck load tests between the

section at chg. 33 + 70 of the main road (Fig. 51) and a section through TR5 shows that the relatively

stiff tree trunks did not deform locally under the wheel loads, while the more flexible underlay at TR5

(5 cm compacted sand between Mirafi 140 fabrics) yielded considerably. Evidently, and not surprisingly,

the stiffer corduroy provided a better load spreading capacity than did the Mirafi 140 sand underlay. One

reason for this is the relatively low elasticity modulus of the Mirafi 140, another the relatively small

thickness of the sand layer. It follows that thin embankments (0.2-0.4 m) consisting of well compacted fill and underlain by

relatively stiff transverse corduroy (slender tree trunks) provide better bearing capacity than do much

thicker embankments consisting of poorly compacted fill and underlain by flexible fabrics and sand.

Trench excavations across the embankments revealed considerable local yielding under a 0.6 m thick

fill on fabrics but no discernible yielding under a 0.2 to 0.5 m road on slender corduroy laid with a few

centimetres spacing. The latter had been subjected to much more severe testing.

6.3 Water-retaining embankments (dams and dykes) on peat over firm subsoil

The writer has found four case records of dykes on peat in Canada (Hardy 1968, Wiesner and Hardy

1978). Of these, only one is founded on peat over firm subsoil. However, the behaviour of the peat

foundation is described or may be assessed separately in each case. In general, it was found that the peat

consolidated relatively rapidly, and the compression of the peat in all cases reduced its permeability to

acceptable values with respect to seepage. Certain objections must be raised with respect to three particular concepts of peat behaviour advanced

in the case records mentioned: (i) the strength of the peat foundation may be assessed on the basis of vane

tests, (ii) the stability of the embankments may be assessed on the basis of "simplified stability analyses",

i.e. undrained failure along a cylindrical failure surface, and (iii) the effective strength parameters may

be determined from triaxial tests. The use of the vane to measure the shear strength of peat was dealt

with in detail in article 5.2.3, where it was concluded that the strength recorded with a vane is under no

circumstances related to the strength governing the behaviour of embankments on peat. The "phenomenal

increase in undrained strength" recorded by Wiesner and Hardy (1978) after the application of 5 to 6 m

of fill is, firstly, an artificial fibre strength and, secondly, certainly is not measured under undrained

conditions. In regard to the use of a $N = 0$ stability analysis for peat, no such failure is known to

have occurred within a peat deposit; the mode of deformation is essentially that of drained vertical

compression, accompanied by some lateral expansion under partly drained conditions. This behaviour

is a direct result of the initially high permeability of the peat, lateral drainage enhanced by horizontal

and possibly vertical hydraulic fracturing in the adjacent unloaded peat, and a lateral internal resistance

induced by the fibres. Exactly the same type of behaviour occurs during triaxial testing, but the internal

lateral (fibre) resistance is unknown and, furthermore, is not included in the lateral effective stress. The

result of this omission is that the apparent angle of friction with respect to effective stresses is much

greater than that determined on the basis of the true lateral stress. This is of particular importance in the

case of dykes on peat, since one critical failure surface could be an essentially horizontal surface within

the lower half of the peat layer, where the pore pressures may be at their maximum. This mode of failure

is better represented by ring shear tests, from which the angle of friction will probably be found to be

very much lower than that obtained from triaxial tests (article 4.3).

Wiesner and Hardy (1978) found that one way to minimize high pore pressure build-ups in peat was

to construct the central portion of the wide fill first, followed by the upstream and downstream portions.

The centre fill was spread and compacted from the centre outwards, the intent being to cause excess pore

pressures to dissipate at a higher rate through lateral seepage in the toe areas and in front of the advancing

fill. This concept is in complete agreement with the writer's findings and is based partly on the fact that

the permeability of the adjacent unloaded peat is significantly higher than that of the compressed peat

beneath the fill, and partly on the observation that hydraulic fracturing in the adjacent peat renders it even more permeable than it was before construction.

6.4 Embankments on peat over soft subsoil

The behaviour of embankments on peat over firm subsoil suggests that the pressure transferred to the

subsoil may be greater than that corresponding to a Boussinesq distribution. Also, since the depth of peat

often is smaller than the width of the embankment, the pressure transferred to the subsoil will not be

significantly smaller than the pressure under the embankment base. It follows that if the peat is underlain

by a soft subsoil, failure may be induced in the subsoil. For example, if the embankment exerts a pressure

on the peat of 30 kPa, the pressure transferred to the subsoil could typically be expected to be in the

range 20-25 kPa. Undrained failure in the subsoil could thus occur if its shear strength was in the range

4-6 kPa, which corresponds to an extremely soft clay. Loads of 70-90 kPa could cause undrained subsoil

failures for a shear strength in the range 14-17 kPa, again corresponding to a very soft clay. As pointed

out by Lea (1964), clay soils below peat deposits may be extremely soft because the unit weight of peat is

so low: beneath 12 m of peat, the effective pressure on the clay may be only 10 kPa. Lea measured vane

shear strengths in a 6 m deposit of soft clay under 6 m of peat in the range 6-16 kPa. He also states that

“in his experience, soft clay under the peat is always present when embankment stability is a problem”. Lea & Brawner (1959) state that “in many instances where peat is assumed to be the cause of highway

distress, the real problem is not the peat itself, but the

very soft soil immediately underlying the peat;

this is true regarding both consolidation movements and embankment stability". A major investigation by MacFarlane & Rutka (1959) of road embankments in about 50 different

muskeg areas led them to conclude that "the cause of shear failures in muskeg areas underlain by a soft

mineral soil layer is due largely to this layer rather than to the peat itself". It is clear from the investigations cited above that failures of embankments on peat over soft subsoil

are not uncommon. These observations also serve to confirm, at least indirectly, the conclusion that

sliding or spreading failures never seem to occur in peat over firm subsoil. The mechanism of the failures occurring in peat over soft subsoil is not known exactly, but it appears

from the description of some of the failures that both rotational and spreading movements have been

observed. Both types of sliding seem to be entirely possible, considering the low strength of the subsoil. In

the case of spreading failure, only the upper part of the clay would be involved in the failure. The possible

failure mechanism shown in Figure 52 is based partly on descriptions of actual failures (e.g. Flaate 1967,

1977) and partly on the knowledge of peat characteristics gained through the present investigation. In Figure 52(a) the active earth pressure in the fill is based on $K_A = 0.3$ and the lateral effective pressure

in the peat is calculated on the basis of $K_0 = 0.2$ and an effective vertical pressure slightly greater than that

corresponding to a Boussinesq distribution. The effective lateral force is thus about 38 kN/m across the

centreline. No excess pore pressure is shown, i.e. Figure 52(a) represents the condition after dissipation

of pore pressures in the peat. The stability conditions would be most critical during construction when excess pore

pressures would

exist in the peat (and in the clay). The effective pressures are therefore smaller, as shown in Figure 52(b).

The lateral effective force in the fill is the same as before (13 kN/m), while that on the peat has been

reduced to about 17 kN/m, giving a total of 30 kN/m. On the other hand, the lateral force across the centreline may have increased considerably as a result

of the excess porewater pressure, perhaps as much as tentatively indicated in Figure 52b. The major force involved in the spreading of the peat thus arises from the excess pore water pressure.

A progressive failure along bd seems therefore entirely possible. At the start of such failure (at b), only

part of T_p would be mobilized, since only a relatively small strain would be required to mobilize the

shear strength of the clay around bc. Also, at this stage it is unlikely that any part of P_p is mobilized. As the clay becomes remoulded or reaches its residual strength, failure would occur progressively

along bd. During this process, P_p would become gradually mobilized, although this would require

considerable horizontal movement of de. At the same time it is probable that the spreading along the

centreline would cause the embankment to fail in the middle. If corduroy is absent, a sinking wedge

could form in the middle, as described by Flaate (1967). If the embankment is supported on corduroy, it

is uncertain what form the embankment failure would take.

6.5 Summary and conclusions

A detailed discussion of various aspects of the construction of roads on peat over firm subsoil is given

by Landva (1980a) under the following headings: (i) behaviour during and after construction, (ii) base

reinforcement, (iii) preloading (surcharge), (iv) embankment berms, (v) drainage ditches, (vi) vertical

sand drains, (vii) light-weight fill, (viii) widening of existing roads, and (ix) behaviour under traffic

loads. Two additional headings are (x) water-retaining embankments (dams and dykes) on peat over firm

subsoil and (xi) embankments on peat over soft subsoil. The reader is referred to the detailed discussions of the various aspects listed above (Landva 1980a).

The following contains a summary and conclusions on these aspects.

(1) A road embankment cannot be suspended or floated on a peat mat. The tensile resistance of the mat does reduce the settlement near the toes and shoulders. However, this effect is temporary since the mat will yield in time.

Figures 52. Stability with respect to spreading failure of embankment on peat underlain by very soft subsoil.

$p' h$ = effective lateral pressure, u = excess pore pressure, T_C = tensile resistance of base reinforcement, T_p = tensile

resistance of peat (≈ 0 at low strain), P_p = passive resistance of adjacent peat. (a) Long-term condition, $u \approx 0$,

spreading failure not probable. (b) Short-term condition, excess pore pressures, spreading failure highly probable.

(2) A spreading failure of the mat is unlikely unless it is underlain by a very soft material (truly amorphous

granular peat without fibres, or very soft clay). In general, the peat under the mat is fibrous, even

if amorphous-granular, and the combination of its frictional behaviour, fibre reinforcement, and low

pore pressures near the top ensures stability against shallow spreading failure. Deeper spreading

failures are, however, possible.

(3) The role of geotextiles as reinforcement is highly

dependent upon the amount of local shear failure

that might occur in the absence of reinforcement (Rowe et al. 1984a). At low road levels (i.e. where

there is a high factor of safety), the geotextile does not alter the embankment behaviour. As the

extent of local plasticity increases (i.e. as the factor safety is reduced), the reinforcement plays an

increasingly more important role in reducing settlement and lateral movements. The major effect of

geotextile reinforcement is to reduce lateral spreading and to increase stability. Even a very high

modulus geotextile will have relatively little effect on consolidation settlement (ibid.). Rowe et al. (1984b) report that the geotextile at the edge of a 5.7 m thick embankment on up to

7.6 m of peat appeared to be unstressed. Also, it was apparent that the use of even a very strong

geotextile did not prevent large shear deformations.

Figures 53. Possible severing of mat at end of rigid base reinforcement. (b) Base reinforcement extended under

berm. Drainage ditch within berm fill.

(4) A relatively rigid base reinforcement, such as corduroy, is of considerable benefit to the behaviour of the embankment, both during and after construction. During construction it will span locally weaker areas and thus distribute the load to give less total and differential settlement and distortions. After construction it will spread traffic loads and thus again reduce differential settlement. All indications are that the embankment thickness can be reduced significantly if the embankment is underlain by corduroy. The termination of the corduroy trunks at the toes of the embankment may cause local shear problems there. However, these may be solved by extending the corduroy under berms (Fig. 53).

(5) Preloading or surcharging the peat appears to be an effective means of reducing future settlement under traffic loads and of reducing wave action. Its effect on the rate of long-term settlement is, however, limited to the rebound

period following removal of the surcharge. The surcharge should be left on until the excess pore pressures have virtually dissipated, i.e. generally about 2-4 months. It is doubtful that much benefit will be realized by leaving it on longer. Conversely, little benefit will be realized if the surcharge is removed when the settlement reaches that expected under the embankment alone. The additional pressure from the surcharge should be at least equal to the maximum pressure to be expected from traffic loads.

(6) Berms have no significant effect on the stability of the embankment. They do, however, provide a smoother transition between the embankment and the peat. When corduroy is used, the trunks should extend at least partly under the berms. The smoother transition provided by this combination is almost certain to reduce difficulties during construction and maintenance after construction. The berms would also serve the purpose of containing shallow drainage ditches required for run-off. Ditch maintenance would therefore also be reduced (Fig. 53).

(7) Vertical sand drains appear to reduce the initial excess pore pressures significantly, but they also appear to become clogged by colloidal peat particles fairly rapidly. This clogging is considered to be a result of the initially high permeability of the peat and the high gradient near the drains. A graded filter may be required to prevent this. The bearing capacity of sand drains is generally not higher than 5-10% of the total load. With respect to traffic loads, however, a combination of direct loading and a transfer of the loads through arching seems to provide greater stability against vibrations and wave action. A single compression test was carried out on SH 3 peat with a vertical slice of sand in the centre of a 229 mm octagonal cell. The compression of the peat and the sand was about 35%, and this resulted in a surprisingly uniform expansion of the sand slice without any tendency of buckling or local shear. Hence it might not be unrealistic to assume that a sand drain when compressed can be represented by the expansion of a cylindrical cavity.

(8) The use of lightweight fill on peat is an obvious solution towards reducing settlement. Sawdust (hogfuel) has been used successfully and fairly extensively, but its present use as fuel will probably prohibit future use. A clear alternative is compressed peat. Since only about 50% compression is required with respect to settlement, it might be possible to compress the peat in situ, e.g. through the use of off-road vehicles equipped with special compression buckets.

(9) The widening of existing roads may result in excessive settlement or failure under the shoulders as

a result of high pressures (and possibly excavation) there. This mode of deformation is also likely to

lead to longitudinal cracking around the centreline. Lightweight fill should be used for at least part

of the new fill at the sides, and any corduroy should be extended. Removal of the mat will only serve

to weaken the peat foundation.

(10) Excess pore pressures are induced instantly by passing traffic loads but they also dissipate instantly,

and little or no residual excess pore pressures remain after the loads have passed. This was observed

for up to about 800 passes of each of four trucks. Thin embankments (0.2-0.4 m) consisting of well compacted fill and underlain by relatively stiff

transverse corduroy (slender tree trunks) provided better bearing capacity than did much thicker

embankments consisting of poorly compacted fill and underlain by flexible fabrics and sand. Trench excavations across the embankments revealed considerable local yielding under a 0.6 m

thick fill on fabrics but no discernible yielding under a 0.2 to 0.5 m road on slender corduroy laid with

a few centimetres spacing. The latter had been subjected to much more severe testing.

(11) The use of undrained strength and vane tests is inapplicable to the stability analysis of embankments,

whether they are road embankments or dams and dykes. In the case of dykes, the strength of peat

measured in ring shear would be a relevant representation of sliding failures. It is also important

that permeability tests simulate field conditions as closely as possible and that the permeability, both

horizontally and vertically, be measured under field stress and strain conditions. The possibility of

hydraulic fracturing exists for the downstream portion of dykes.

(12) Most, if not all, shear failures of embankments on peat occur as a result of soft subsoil underlying

the peat. Both rotational and spreading failures are possible. In the case of spreading failures the

outward movement is slowed down and possibly arrested by the gradual mobilization of the passive

resistance in the peat outside the toes and by a decrease in excess pore pressures as the spreading

develops. The spreading failure is caused by high excess pore pressures in the centre and could be

prevented therefore by the installation of properly designed vertical sand drains and, if necessary, by

stage loading. It is noted that a spreading failure is likely to result in the same surface geometry as

a rotational failure, i.e. bulging of the peat outside the toes accompanied by some lateral movement,

and a sinking of the central portion of the fill. A spreading failure could therefore be mistaken for a

rotational failure.

7 GENERAL SUMMARY AND CONCLUSIONS

7.1 History

The technique of spreading the load on the peat by means of corduroy was known and used as early as about

4500 years ago. A combination of corduroy and light-weight fill (peat) was used in Ireland (Mullins &

Mullins 1848) in the early part of the nineteenth century. Settlement of the road was effectively reduced

in advance of embankment construction by drawing down the

ground water level through long-term (two years or more) drainage to progressively lower levels, combined with the application of additional load from the peat spoil. This method of drainage (draw-down) and loading was used also in the construction of canals through peatland. The stability of the canals, both upon flooding and in the long term, was ensured through proper attention to the action of seepage forces. Corduroy (cross-logging) and light-weight fill were used for railway embankments in Canada before the turn of the century (Mackenzie 1904). Corduroy was not used if the bank was high and settlement was likely to be considerable, in which case it was considered to do more harm than good (Fig. 53).

Materials used for light-weight fill included turf, peat, sawdust and cinders. It appears therefore that the use of sawdust as light-weight fill originated in Canada before the turn of the (20th) century. Despite the development of the science of soil mechanics in the last almost one hundred years, and despite vast improvements in road design tools and construction equipment during the same period, there have been no conspicuous improvements in the design and construction of embankments on peat. In fact, the methods applied some 160 years ago in Ireland seem rather superior to those practised today.

7.2 Peat and peatland

Peatland soils were divided into two groups by von Post (1922), viz. sedimentary and sedentary peats.

Sedimentary peats include gyttja and other organic soils which may have a high mineral content. Seden

tary peats consist mostly of remains of vegetal matter, but may also contain some mineral matter deposited

by wind or flood waters. The peats discussed in this paper

belong to the sedentary group. One outstanding characteristic of peats

within this group is their fibre content, others are their generally very high porosity, permeability and

compressibility. These properties do change with the degree of humification, but are still conspicuous

even if the peat appears to be almost completely humified (H 9). The sedimentary peats are better described as organic soils. Their generally plastic behaviour is quite

different from that of sedentary peats and is in fact more like that of soft plastic clays. These peats are

not discussed in any detail in this paper. In fact, on reviewing case records pertaining to peat, it was

necessary to exclude many records which proved to pertain to sedimentary peats (organic soils). In many

cases the peat description given was not sufficient for a distinction to be made even between these two

broad groups. Since peatland soils are now generally referred to as peat and since a peat may therefore be any

material between two such extremes as a coarse-fibrous matting and a jelly-like gyttja with 80% clay

and silt, it follows that it is extremely important to describe the peat in detail, as in the case of mineral

soils. A case record of construction on peatland is of little value without such detailed description of the

peat(s) involved.

7.3 Fabric and structure

Microscopy investigations, including scanning electron microscopy, of two sedentary peats (sphagnum

and sedge peats) showed that the peat particles are highly porous at all stages of humification. Pore

pressures induced in peat are not therefore higher within the particles than between the particles, and

pore water flow cannot therefore be separated into flow between the particles and flow out of the particles

into interparticle voids. In fact, at the very common degree of humification of H 3 (and higher), the

perforated particle membranes are partly or completely decomposed and the peat structure is therefore

partly or complete open. The peat particles are generally flat and elongated, even at high degrees of humification, and this

fibrous structure provides a reinforcement in the same manner as that occurring in reinforced earth. This

reinforcing action is generally present even at very high degrees of humification, because the highly

humified matrix is nearly always interwoven with more resistant fibres of lower humification. Thus a

truly amorphous-granular peat without fibres is quite rare. Since the degree of fibrosity (La Rochelle 1980) in many respects governs the behaviour of the peat

under load, it is important to study the fabric and structure of a peat in connection with geotechnical

investigations. Scanning electron microscopy has proved to be a valuable method of investigating fabric

and structure. Rowe's (1972) method of fabric investigation would also seem applicable for this purpose.

7.4 Geotechnical behaviour

The geotechnical behaviour of (sedentary) peats is characterized by their - high initial porosity - high initial permeability - high compressibility - high rate of creep - generally high degree of fibrosity

and by changes in these characteristics with time and applied pressure. Although it is possible to simulate

field behaviour more or less accurately in the laboratory on undisturbed samples, the test results are often

not representative of field conditions. This is directly attributable to the considerable variability of peat

in the horizontal as well as in the vertical direction. This variability may exist even if the index properties

and appearance of the peat suggest that the peat is relatively uniform. The geotechnical peculiarities of peat are a direct result of its high porosity in combination with the

high flexibility and plasticity of the organic solids and their tensile strength. Thus the porosity and the

flexibility give rise to a high compressibility, a high permeability and a very low unit weight. The plastic

nature of the solids gives rise to a high rate of creep, and the tensile strength of the elongated solids

provides a reinforcing action in certain modes of deformation, such as compression. The changes in geotechnical properties with time and pressure may be compared to those in a fibrous

material such as felt or fibreglass matting. As the material is compressed, the porosity and hence the

permeability decrease, and the compressibility also decreases. As long as the tensile strength of the fibres

is not exceeded, failure of the material is effectively prevented by the lateral internal resistance induced

by the fibres, provided that the pore pressures are relatively low. For example, it is simply not possible to

produce failure in felt or fibreglass matting by applying a load perpendicular to the direction of the fibres.

The material is simply compressed vertically, and its lateral expansion is insignificant. This behaviour

is also typical of peat, but some lateral expansion does of course occur on account of the pore pressures

induced by the load and the relatively low tensile strength and high ductility of the particles. It is noted

that the applied load need not in fact be perpendicular to the prevailing direction of the fibres. If it is

parallel to the fibres, some lateral expansion will occur initially until the fibres, through stress induced

anisotropy, are more or less flattened in a direction parallel to the plane of the major principal stress.

The lateral internal resistance induced by the fibres is an unknown quantity and is not normally included

in the lateral effective stress determined from triaxial test results, whether drained or undrained. Since

the applied lateral effective stress is lower than the real stress, the calculated angle of friction is generally

much too high. If the mode of deformation in the field is identical to that in the triaxial test, this may not

be important. However, if the mode of deformation is different, such as that of horizontal sliding (e.g.

in the case of dykes), the reinforcing action of the fibres is insignificant. The error involved in using the

results of triaxial tests for such a case is thus on the unsafe side. Typical values of friction parameters

for the peat obtained in triaxial testing are $c = 0$ to 5 kPa, $\phi = 40^\circ$ to 50° , while those in ring shear for

the Escuminac peat were $c = 2$ to 3 kPa, $\phi = 27^\circ$ to 33° .

The rate of creep in peat is often considered to be proportional to the logarithm of time and to increase

with applied stress. This assumption is not generally valid. In many cases this proportionality does not

exist at all, and in other cases the rate of creep is not affected by the magnitude of applied stress. It

may be difficult, if at all possible, to predict the in-situ rate of creep on the basis of laboratory tests,

particularly if small samples are used. One particularly interesting feature of peat is that its geotechnical behaviour does not seem to change

significantly with the degree of disturbance of the structure. For example, the shear strength as measured

in ring shear and the compression measured in consolidation rings were not affected even if the peat

samples were first completely remoulded and consolidated from a slurry. The reason for this is evidently

the stress-induced structure (anisotropy) of the peat, as noted earlier. The initial porosity of undisturbed

peat is so high that the compression of the peat results in approximately the same structure whether it is

compressed from its undisturbed state or from a slurry.

The classical theory of consolidation is not applicable to peat owing to the high rate of creep (secondary

consolidation) in peat and the change in permeability with time and applied pressure. A modified theory of

consolidation (Berry and Poskitt 1972) taking these changes into account suggests that a combination of

a high rate of creep and a relatively low permeability could result in the generation of pore pressures.

Although this mechanism would be more pronounced in sedimentary peats, where local pore pressures

thus might not dissipate at all for a very long period of time, the observation of residual pore pressures in

sedimentary peats suggests that pore pressures may also be generated in this type of peat, although to a

lesser extent on account of a higher permeability.

The low unit weight of peat readily lends itself to the occurrence of horizontal hydraulic fracturing.

This is particularly evident in the case of embankments on peat, where the pore pressures induced in

the loaded peat zone readily produce horizontal fracturing due to outflow from the loaded zone into the

adjacent unloaded zones.

The vertical effective pressure in the peat before loading

may be low or even zero. The pressure applied

by a narrow embankment decreases with depth, and if the peat is underlain by a relatively impervious

material, there will be a temporary downward flow. At the same time, a lateral flow will also occur

towards the unloaded peat adjacent to the embankment. A period of constant excess pore pressure is thus

possible in the lower part of the peat. Indications are that the horizontal permeability of peat is higher than the vertical permeability, particu

larly in highly humified peats. Consolidation in the field may therefore be much more rapid than that

which would be expected from standard consolidation or permeability tests in the laboratory.

7.5 Geotechnical exploration

Since structures on peat generally cover an extensive area of peatland, it is necessary to conduct a rea

sonably extensive, but simple, survey of the area prior to any detailed investigation. The information

obtained from such a survey includes peat types and depths, cover (mat) types, physical characteristics,

and water regime. It is of importance to the geotechnical engineer that the peats be properly identi

fied, including the nature and extent of fibres and other reinforcing agents (roots, root hairs, etc.). The

preliminary survey should also include disturbed sampling of the subsoil. Detailed regular geotechnical field testing of sedentary peats is not generally feasible. The only regular

type of test that may give meaningful results is the plate load test, but this is limited to the strength of

the mat or to a combination of the resistance of the mat and the underlying peat. Plate, cone or vane

tests in peat are affected significantly and to an unknown

degree by fibre action and cannot therefore

give any proper information on the stress-strain characteristics governing the behaviour of the peat under

embankments or in excavations. Undisturbed sampling of moss peats is readily accomplished with the 100 mm diameter piston sampler

described in chapter 5. Relatively heavy accessory equipment is required and the use of an off-road vehicle

is mandatory. The 100 mm sampler was developed especially for sedentary peat, but it is equally applicable in

sedimentary peat and other extremely soft, almost fluid-like soils. However, this sampler proved to defy

any attempt to obtain samples in a blanket bog containing sedge peat. Despite a sharp cutting edge and

despite tapping rather than pushing the sampler into the peat, it acted more or less like a closed-end tube,

and very little peat entered the sampler tube. The order of magnitude of expected settlements may be estimated on the basis of consolidation tests,

preferably on relatively large samples (e.g. not less than 100 mm diameter). The rate of long-term

settlement (creep) in the field does not, however, seem to readily lend itself to laboratory simulation. The shear strength of sedentary peats is not normally a critical factor. However, in the case of horizontal

sliding, such as might occur under a dyke, the shear resistance may be investigated by means of ring

shear tests. A special apparatus is then required to allow for the high compressibility of the peat. The behaviour of the peat under a road embankment can be investigated by means of triaxial tests with

controlled lateral strain or by plate tests in a large cell where the mode of deformation can be observed and

measured during testing. The results of such tests may not be strictly applicable quantitatively because

of the variability of the peat. However, a qualitative assessment of the expected behaviour may prove

very useful in the design of the embankment. Permeability (hydraulic conductivity) tests should be carried out under different effective stresses.

It is important that the seepage pressures be included in the calculation of the effective stresses, since

typically the differential head used in laboratory testing may constitute a significant portion of the

pressure representing the weight of an embankment. The permeability should be measured both in the

horizontal and in the vertical direction.

7.5.1 Embankments on peat

Geotechnical investigations of peat and construction on peatland almost always pertain to embankments,

mostly road or railway embankment, but also pipelines and dykes. In all cases large settlements are

involved, in some cases accompanied by significant lateral displacements, and in some cases distinct

shear failures occur. Opinions vary therefore with respect to the design of embankments on peat. The present investigation has shown clearly that any discussion of embankments on peat, particularly

with respect to the design of the embankments, must include a clear definition and description of the

peat and the subsoil. Thus a distinction must first of all be made between sedentary and sedimentary

peats, since the properties of these two groups may be as different as those of gravel and soft plastic clay.

Furthermore, it is necessary to distinguish between firm and soft sub-soils, since shear failures often

occur in soft subsoils. If these distinctions are made, it becomes clear why different investigators hold

different opinions on the design of embankments on peat.

The peats discussed in this paper belong to the sedentary group, i.e. mostly fibrous peats with varying

content of fibres. Also, mostly embankments on peat over firm subsoil have been considered, although

a brief discussion pertaining to soft subsoils has also been included. No conventional type of shear failure is likely to occur in the case of embankments on (sedentary)

peat over firm subsoil, regardless of the thickness of the embankment. No floating or suspension of the

embankment on the peat cover (mat) is possible, and any resistance offered by the cover is temporary. A

reasonably rigid base reinforcement is of significant benefit with respect to increased bearing capacity and

differential settlements. The reinforcement should be extended outside the embankment under additional

fill (berms). This provides a smooth transition, and the berms can also contain the drainage ditches.

Geotextiles may be useful for the prevention of local failures under very coarse fill. The relatively rapid dissipation of pore pressures in the peat is a result of the high permeability of the

peat and of hydraulic fracturing in the peat adjacent to the fill. The temporary heave of the peat adjacent

to the fill is a result of horizontal hydraulic fracturing, i.e. inflow of water and gas from the loaded peat

into the adjacent unloaded peat zones. Preloading of the peat by application of a surcharge will reduce settlement under traffic if the additional

pressure is equal to or greater than that under the traffic loads and if the surcharge is left on until the

excess pore pressures have virtually dissipated.

Vertical sand drains reduce the initial excess pore pressure but have limited effect on the rate of

settlement and pore pressure dissipation. This limited effect is considered to be a combined result of

clogging of the drains and lateral compression of the peat under the high initial hydraulic gradients.

Vertical sand drains carry an insignificant portion of the embankment load, but they do appear to absorb

a significant portion of traffic loads, either directly or through arching.

A widening of existing road embankments on peat may result in considerable differential settlements

and longitudinal cracking, since normally the additional loads will be greater at the shoulders.

The obvious solution to settlement problems is to use as light an embankment as possible. Indications

are that compressed peat bales may be the most economical light-weight fill. About 50% compression of

the peat would be sufficient, and this may be accomplished in situ, either in connection with the cutting

of the peat fill material or immediately after placing. A combination of well compacted mineral fill,

compressed peat and a relatively rigid base reinforcement (corduroy) should provide optimum use of

materials and relatively low maintenance costs. In the case of embankments on (sedentary) peat over soft subsoil, the embankment loads transferred

to the subsoil may cause spreading or rotational failures there. The analysis of such failures would be

carried out on the basis of methods established for mineral soils. It is noted however that the subsoil

may be extremely soft as a result of very low effective stresses, and that the behaviour of the peat will to

some extent affect the failure mechanism involved. Embankments on sedimentary peat (organic soils) have not been considered in this paper.

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APPENDIX A

The modified von Post classification as used in this paper (von Post 1922; Kivinen 1948; Briaud 1974;

Korpilaakko and Woolnough 1977) is as follows:

1. Genera Bryales (moss) = B Carex (sedge) = C Equisetum (horse tail) = Eq Eriophorum (cotton grass) = Er Hypnum (moss) = H Lignidi (wood) =W Nanolignidi (shrubs) = N Phragmites = Ph Scheuchzeria (aquatic herbs) = Sch Sphagnum (moss) = S

2. Designation With few exceptions natural peats consist of a mixture of two or more genera. The designation adopted in this paper is to list the genera in descending order of content, i.e. the first symbol represents the principal components. For example, a peat classified as ErCS (Fig. 22, bottom layer) consists mainly of Eriophorum remnants, while the content of Carex remnants would be lower and that of Sphagnum remnants relatively low.

3. Humification (H) The degree of humification is graded on a scale from 1 to 10 and designated H 1 to H 10 . The various degrees of humification are recognized as shown in Table A1.

4. Water Content (B) In the field the water content of the peat is estimated on a scale from 1 (dry) to 5 (very high), designated B 1 to B 5 . In terms of actual water contents the following ranges are suggested: B 2 < 500%, B 3 500-1000%, B 4 1000-2000%, and B 5 > 2000%.

5. Fine Fibres (F) Fine fibres are defined as fibres and stems smaller than 1 mm in diameter or width. They are often of the Eriophorum genus, but Hypnum and Sphagnum stems may also be included if properly specified, e.g. F(H) or F(S). Shrub rootlets may also be included, specified as F(N). No special designation is indicated for plant root hairs, rhizoids, or other fibres. The content of fine fibres is graded on a scale from 0 to 3 as follows: F 0 nil, F 1 low content, F 2 moderate content, and F 3 high content.

6. Coarse Fibres (R) Coarse fibres are defined as fibres, stems, and rootlets greater than 1 mm in diameter or width. They are often of the Carex genus, but Hypnum and Sphagnum stems may also be included if properly specified, e.g. R(H) or R(S). Shrub (N) rootlets are specified as R(N). The content of coarse fibres is graded on a scale from 0 to 3 as follows: R 0 nil, R 1 low content, R 2 moderate content, and R 3 high content.

7. Wood (W) and Shrub (N) Remnants Wood and shrub content is graded on a scale from 0 to 3 as follows: W 0 nil, W 1 low content, W 2 moderate content, and W 3 high content. The original designation for macroscopic wood (W) and shrub

(N) remnants was, collectively, V (from Swedish vedrester = wood remnants, von Post 1922).

Table A1. Degree of humification (von Post system). Content of Material extruded

Degree of Plant amorphous on squeezing (passing

humification Decomposition structure material between fingers) Nature of residue

H 1 None Easily identified None Clear, colourless water

H 2 Insignificant Easily identified None Yellowish water

H 3 Very slight Still identifiable Slight Brown, muddy water; Not pasty no peat

H 4 Slight Not easily Some Dark brown, muddy Somewhat pasty identified water; no peat

H 5 Moderate Recognizable, Considerable Muddy water and Strongly pasty but vague some peat

H 6 Moderately Indistinct Considerable About one third of strong (more distinct peat squeezed out; after squeezing) water dark brown

H 7 Strong Faintly High About one half of recognizable peat squeezed out; any water very dark brown

H 8 Very strong Very indistinct High About two thirds of Plant tissue capable of peat squeezed out; also resisting decomposition some pasty water (roots, fibres)

H 9 Nearly complete Almost not Nearly all the peat recognizable squeezed out as a fairly uniform paste

H 10 Complete Not discernible All the peat passes between the fingers; no free water visible.

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Characterisation and Engineering Properties of Natural Soils - Tan, Phoon, Hight & Leroueil (eds) © 2007 Taylor & Francis Group, London, ISBN 978-0-415-42691-6

Geotechnical characterization and behaviour of Argentinean collapsible loess

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ABSTRACT: The Loess formation in Argentina is the largest windblown deposit in the southern hemi

sphere. The most modern loess deposits are characterized by

an open structure made of fine sand and

volcanic silt particles weakly bonded. Unsaturated clay
buttresses, soluble salts, and some others less sol

uble cementing agents (e.g. carbonates, gypsum, iron oxide)
are the main binders. The combined effects

of suction and cementation stabilize the soil structure at
a very high void ratio. Thus, in natural state,

Argentinean loess is capable to withstand moderate external
loads and even vertical slopes. However,

severe erosion or massive collapse may occur as the soil
comes in contact with water. Below the yielding

stress (collapse), the soil behaves as a linear elastic
material and the stiffness is governed by saturation

and degree of cementation. At higher stresses, the collapse
potential of the soil skeleton is governed by

a complex interplay between applied external pressure and
internal forces due to cementation, matric

suction and coulombic attraction-repulsion forces developed
at colloidal particles size. Capillary forces

and cementation cause an increase in stiffness, yielding
stress, shear strength, and tendency to stress

localization. At small strains and low confining pressure,
loess behaves as a structured, cemented soil.

At large strains and high confining pressure the initial
structure of the soil is destroyed. Foundations in

deposits of loess require the use of soil improvement
techniques or deep foundations.

1 INTRODUCTION

The term "loess" was originally used in Germany to specify
the soil formation located in the Rhine basin

(notice that it has the same root as the word "loose" in
English), and refers to a windblown deposit of silty

soil characterized by an open structure. Although it can be

controversial, the original (primary loess)

when re-transported and re-deposited by means of a different atmospheric agent is designated here as

secondary loess. Approximately 10% of the continents are covered by loess, with the most significant

deposits being in North America, Europe, Asia and South America.

The first description of loess in Argentina was by scientists in mid-eighteenth century, who made

a morphological comparison with European deposits. To the knowledge of the authors, Castellanos

(1962) and Frenguelli (1957) prepared the first comprehensive geological description of Argentinean

loess formations. These are found in the east Pampas of the country and in several small valleys. The

distribution of loess deposits and other windblown deposits in Argentina are shown in Figure 1. The

largest deposit of loess is located in between parallels 23 ° and 38 ° South (Teruggi 1957). The parallel

30 ° S was recently proposed to divide this deposit into the Pampean-temperate and Chaco-neotropical

loess (Sayago 1995, Zarate 2003). Aeolian sands (sand sea) or alluvial deposits cover the rest of the

pampas toward the south (Iriondo 1999). The Pampean-temperate loess covers more than 600,000 km²

along a band of 2000 km in length and 250 to 300 km wide surrounding the Pampean Sand Sea. There are

other loess deposits in some valley of Tucuman, the high plain lands of Cordoba, San Luis, Catamarca

and the piedmont of Mendoza. Loess deposits have also been observed in the neighboring countries:

Paraguay, south of Brazil, and Uruguay.

The thickness of the loess to the north east of Buenos

Aires, is around 10-15 m (Gonzalez Bonorino,

1966, Fidalgo 1990, Iriondo 1990) and increases towards the west. It is up to 40 m thick to the west of

Buenos Aires (Rabassa, 1990) and at the foot of the Sierras Pampeanas of Cordoba (Sayago et al 2001).

Figure 1. Distribution of Loess and windblown deposits in Argentina (modified from Zarate 2003).

In Andean subtropical valleys (Tucuman) it reaches 20 to 60 m in thickness (Collantes et al. 1993 , Zinck

and Sayago 1999). In the western Chaco plains, the loess deposit has an average thickness of 10-20 m

(Bonaparte and Bobovnik 1974, Esteban et al, 1988, Sayago et al, 2001). Towards the center and east of

Chaco plains, the loess reaches a maximum thickness of 10 m (Ledesma et al. 1973). In the engineering field, Argentinean loess was originally mentioned in the classic book of Scheidig

(1934), but, the fundamentals of its geotechnical behavior was only described several decades later

(Bolognesi and Moretto, 1957, Reginatto, 1971, Nuñez et al. 1970; Bolognesi, 1975; Moll, 1975; Moll,

et al.1988; Moll and Rocca 1991). All these studies differentiate the primary loess from the reworked

secondary loess in the Pampean-temperate deposit (see Figure 1). The primary loess is the more recent

deposit, characterized by a highly collapsible structure. The collapsible structure is the result of a naturally

unsaturated condition, weak cementation, poorly accommodated and open fabric, uniform particle size

distribution and a high proportion of platy particles. The secondary loess has been re-worked and behaves

Figure 2. Effect of water on primary loess: a) Erosion of a channel, b) Destructive settlement of a building,

c) Sinkholes, and d) Land sliding.

as a silt or clay with a relatively stable structure.
Figure 2 displays some pictures recorded in different

places of Argentina that show some of the main engineering
problems that occur when water interacts

with primary loess.

This paper presents a comprehensive review of some
fundamental aspects related to the physical

properties and engineering behavior of primary loess. The
work briefly summarizes the origin, formation,

and distribution of loess in Argentina. The soil structure,
fabric and mineral composition are described

and related to soil collapsibility. The stress-strain
behavior of undisturbed loess specimens, including

the role of suction and cementation, is discussed based on
a large number of laboratory tests performed

at different strain levels. The main findings related to
the electrical properties of loess, as measured at

various frequencies, are also included herein. Finally, the
authors highlight some of the geotechnical

solutions adopted in engineering practice in the design of
foundations on loess.

2 ENGINEERING GEOLOGY

The north of the Patagonia has been considered the origin
of the Pampean loess since the beginning of

the 20th century, although it was Teruggi (1957) who gave
the first comprehensive explanation of loess

formation. Recent studies (see for example Zarate, 2003),
emphasize that although Teruggi's model

correctly describes the origin of the South Pampa deposit,
he is not able to explain the sedimentological

variations in the North Pampas and Chaco deposits.

Aeolian soils has been deposited from the Upper Miocene to

the Upper Holocene. The oldest deposits have been related to an orogenic phase of the Andes in the Upper Miocene (ca 10 My) which dried the humid airstreams coming from the Pacific Ocean generated the accumulation of several hundreds of meters of sediments. These deposits actually include extensive areas of Pampas, along the Piedmont of Mendoza and in the northwest (Catamarca, the Rioja and Tucumán). This sedimentation was predominantly pyro-clastic and today this soil is found at the base of the loess profile.

The primary source of materials of the youngest Pampean loess are silt and fine sands formed by physical weathering in snowy environment of the Andes Mountains located to the north of latitude 28 ° S. These sediments were transported by glacial waters to the south, along the piedmont of the Andes Mountains, by means of the Desaguadero fluvial system (see Figure 1). A significant volume of the sediments was deposited in a wide flood plain and in large alluvial cones between the latitudes 37-38 S.

These deposits were thereafter transported by winds from the S-SW of Patagonia, as shown in Figure 1.

The sandy particles were carried by saltation forming the Sand Sea in a dry-desert environment. Towards the North East of the Sand Sea, in a semi-desert climate, the winds resulted in the deposition of fine dust in suspension, forming the belt of loess sediments adjacent to the sand sea (Krohling 1999). The modern units are predominantly clastic (silts, sand, loess). Today, it is believed that there are several other deposits of loess originating from mechanisms of multi-stage transport (Zarate 2003). Several alluvial fans partially interbedded and covered by loess

deposits have been reported along the piedmont of the Sierras Pampeanas of Cordoba and San Luis.

The Sierras Pampeanas apparently constitute a secondary source of the igneous and metamorphic rock

particles usually found in the loess. Ash rains from the Andean eruptions also constitute an important particle source for South American

loess. It has been estimated between 5 to 10 m of pyroclastic material has accumulated in the last one

million years. Mineralogical data indicate that around a 10% of loess of the Late Pleistocene-Holocene was

deposited by this mechanism. Finally, it is important to mention that along the parallel 40 ° S, W-E humid

winds allowed for the transformation of minerals to halloysite and finally smectite and vermiculite. In the last decade numerous data on Pampean and Chaco loess deposits have shown dates from the

Late Pleistocene to the Holocene (Sayago et al. 2001). The results suggest periods of deposition between

10 and 30 ky b.p. that coincide with peaks in the thickness of Antarctic ice. Although the deposition of

loess has not been recorded in the Altithermal period (8 to 6 ky b.p.), both in Pampas and in Chaco, there

are several episodes of deposition dated to the Middle and Upper Holocene. In general, the thickness of the youngest Holocene loess of the Pampean and Chaco formations

(primary loess), is less than 5 m. Sometimes it is difficult to differentiate it from loess of the Late

Pleistocene (secondary loess), as the deposits have similar color, physical characteristics and structure.

It should be also pointed that in some areas showed in Figure 1 the primary loess is absent. The history of the loess deposits controls some of their geomechanical properties. The modern

Holocene loess (also known as Cordobense Formation) is very

loose and self-subsident. The pres

ence of volcanic ash can result in a pouzzolanic process generating a cemented silt, locally known as

tosca. The oldest loess (late Pleistocene/Holocene) which has been removed and reworked by water,

behaves as silt of varying dry density. The oldest of the loess deposits near the Parana and La Plata

rivers suffered the influence of glaciations. In the late Pleistocene, the sea level dropped by more than

100 meters, causing a depression in the phreatic level and so overconsolidating the soil by desiccation

(Bolognesi 1975).

3 COMPOSITION

The particle size of the loess decreases with distance to the original material source. Table 1 shows the

range of grain size commonly encountered in primary Pampean loess. On the central area of Argentina,

silt is the most abundant fraction. Carbonates are usually found dispersed as a cementing agent, and also

as nodules which may be larger than 20 mm in size. The mineralogy of Pampean loess (see Table 2) is characterized by an abundance of plagioclase

(20-60%), relatively little quartz (20-30%) and a significant percentage of volcanic glass (15 to 30%)

(Teruggi 1957, Zarate and Blassi 1993, and Krohling and Iriondo 1999). The Holocene Loess shows the

influence of post-depositional morpho-dynamic processes. Examples of the results of these processes are

the crystalline minerals encountered from the Sierras Pampeanas of Cordoba, and mineralogical compo

nents derived from the Brazilian plateau in the loessic substrate of soils of Santa Fe (northeast Pampas)

that are probably due to sources in the old plains of the

Parana River. Illite appears as the dominant

clay mineral in the Middle Pleistocene to Holocene sequences, while smectite and kaolinite are minor

components. Montmorillonite predominates in the Pampean loess dating from the Lower Pleistocene. The most significant difference between the Argentinean loess and other loess formations around the

world is the presence of volcanic glass minerals as a major component of the silt and sand particles

(Teruggi, 1957, Moll and Rocca, 1991). The content of calcium carbonate usually ranges between 2 and 10%. It is present in the form of nodules

or precipitated at particle contacts. When gypsum and iron oxides are present, the soil is very stable

even in the presence of water. Loess is alkaline in nature ($\text{pH} > 8$). The most abundant ions in solution

and adsorbed are sodium and calcium (see Table 3). The specific surface area is mainly controlled by

the clay fraction and ranges from $1 \text{ m}^2/\text{gr}$ to $10 \text{ m}^2/\text{gr}$.
Table 1. Range of grain size distribution commonly encountered in Pampean loess. Grain size Percent by weight
Sand ($>0.1 \text{ mm}$) 1.5-10 Silt ($0.1 \text{ mm}-0.005 \text{ mm}$) 40-80 Clay ($<0.005 \text{ mm}$) 20-35
CO 3 Ca 2-10
Table 2. Range of mineral components for the different fraction (from Sagayo et al. 2001). Particle size Mineral Percent by weight(%)
Sand Quartz 20-30 Feldspar 20-65 Volcanic Glass 1-25 Basalt 1-25
Silt Quartz 20-30 Feldspar 30 Volcanic Glass 15-60 Gypsum 20
Clay Illite 90-100 Montmorillonite 0-5 Kaolinite 0-5

Table 3. Range of exchangeable and soluble cations in loess.

Exchangeable cations (me/100 g) Soluble cations (me/100 g)

Ca Mg Na K CEC Ca Mg Na K

3-12 2-6 1-15 1-3 14-25 0.1-0.5 0.06-0.2 0.5-4 0.03-0.1

4 STATE AND INDEX PROPERTIES

Table 4 and Figure 3 summarise a parametric study of the index properties of loess, obtained from more

than 420 boreholes in Cordoba City. In general, the plasticity index and liquidity index of primary loess

(upper layers) are very low (less than 6%) and increases with depth for secondary loess layers. The

soil classifies as ML or CL-ML in the unified classification system. The dry unit weight ranges from

12.5 kN/m³ to 14.5 kN/m³ and the average specific gravity is close to 2.65. The natural water content

varies, mainly with depth, from 12% to 15% at the surface to over 25% at a depth of 20 m. The maximum

dry unit weight obtained from the standard Proctor test is typically around 16.5 kN/m³ and 17.5 kN/m³

for the optimum water content, which varies from 16% to 17%.

The matric suction for undisturbed samples of primary loess is displayed in Figure 4, for the range of

water contents typically found in nature. Tests were performed in a suction cell by means of the suction

translation axis method (Fredlund and Rahardjo, 1993). In the suction curve of loess, the characteristics

points of air entry pressure is 10 kPa and residual volumetric moisture is 0,10.

5 FABRIC

Figure 5a shows scanning electron microscope (SEM) photograph of the fabric of loess around a macro

pore. A detail of the structure is shown in Figure 5b. Clay minerals and precipitated salts are observed on

the surface of the silt and sand particles. Clay minerals, especially illite and montmorillonite, may have

formed after deposition from the chemical weathering of the volcanic glass and plagioclase minerals.

There is no clear evidence of clay bridge structures connecting the largest particles, which has been

Table 4. Range of variation of main physical parameters of

Pampean loess obtained from 420 boreholes.

Litological ω ω_L IP PPS4 PPS10 PPS40 PPS200 γ_d

identification (%) (%) (%) (%) (%) (%) (%) (kN/m³)

LC 2A

Average 16.08 24.25 4.58 99.93 99.49 96.98 90.64 12.9

Standard deviation 5.15 2.84 1.93 0.50 1.81 4.95 10.53 1.07

LC 2Ax

Average 17.37 24.50 4.84 99.62 99.34 95.48 87.82 13.9

Standard deviation 4.74 3.32 1.76 3.11 1.58 6.51 15.22 5.00

LC 2B

Average 22.07 26.54 6.71 99.79 98.54 93.39 84.17 14.8

Standard deviation 7.78 5.34 3.52 1.04 3.13 13.00 15.55 8.33

LC 2C

Average 28.66 35.02 10.86 100.00 99.63 96.61 88.55 13.4

Standard deviation 11.16 10.40 6.77 0.00 0.86 6.27 9.87 1.17

Note: Litology Complex LC 2A refers to primary loess, LC 2Ax to carbonatic cemented loess, LC 2B to

secondary loess from the Upper Pleistocene and LC 2C to the oldest loess from the Middle Pleistocene, ω is

natural water content, ω_L is the liquid limit, IP is the plastic index, PPS is the percent of soil passing sieve

followed by the respective sieve number and γ_d is the dry unit weight. 0 5 10 15 20 0 10 15 20 25 30 35 40 45 50 55
Liquid Limit (%) P l a s t i c L i m i t (%) Primary
Loess Secondary Loess 2B 2C Line A Line A 5

Figure 3. Atterberg limits of primary and secondary loess.
0.00 0.05 0.10 0.15 0.20 0.25 0.30 0.35 0.40 0.45 0.50 1 10
100 1000 Matric Suction (kPa) V o l u m e t r i c W a t e r
C o n t e n t Blundo (1996) Redolfi (1990) Zeballos (1997)
Saturation

Figure 4. Variation of matric suction of primary loess with respect to volumetric water content.

Figure 5. SEM photograph of a macropore in the structure of loess, b) SEM of sand and silt particles around the

macropore (Rinaldi et al. 2001). macropore macropore Sand particles 200

Figure 6. Fabric of the Argentinean loess - Sketch based on SEM images.

observed in other loess deposits (see for example Kie, 1988). Instead, coarser particles seem to accom

modate around macropores in a disordered manner. The presence of clay minerals and other cementing

agents is observed at contact points. Figure 6 shows a sketch of the structure interpreted from SEM

photographs of Pampean loess. The average porosity of the loess is around 0.5. Submicroscopic pores

(<1 μm) represent from 5% to 25% of the total pore volume, microscopic pores (1-20 μm) represent

from 30% to 80% of the total pore volume and the remaining are macropores of the millimeter size

usually coated by a thin layer of carbonates (Luttenegger and Saber, 1987; Barton, 1994).

Usually loess has some degree of weak cementation provided by soluble salts, amorphous silica,

calcium carbonate, gypsum and iron oxide. The strengthening of the soil structure due to cementing

agents is sometimes difficult to perceive since most of the soils weaken in the presence of water. Iron

oxide, silica amorphous and carbonates are usually the most stable cementing materials. Large amount

of cementing agents fully disseminated in the soil and precipitated at particle contacts can result in a

significant increase in the shear strength of loess, to the extent that it may behave as a sedimentary rock.

An example is that known locally as “tosca”. (Rocca, 1985, Quintana and Redolfi 2001). More localized

cementation results in the formation of nodules and aggregates of particles. In this case, loess behaves

like a soil with a larger particle size, and its stress-strain behavior is controlled mainly by the size and

distribution of the aggregates in the soil mass.

Figure 7a displays the grain size distribution curve of a structured specimen obtained from a block

sample recovered from 1 m depth open trench. Water table at this place was at 20 m depth. The sieving

analysis was performed following the conventional sieving test (ASTM D 422), but with some changes

as outlined below. A block of the structured sample was placed on the coarsest sieve of the series and

gently washed until the water extracted from last sieve No 200 (0.075 mm) become clear. This showed

that there were no more fine particles being detached from the aggregates retained in the upper sieves (a) (b)

Particle Diameter Fully 0110

%

P

a s

s i n

g Fully Destructured 0 1 2 3 4 5 6 7 8 9 10 Structured

Figure 7. a) Sieve analysis performed on structured and fully destructured loess specimens, b) cemented aggregates

obtained from sieving analysis of a loess sample without deconstruction of the soil by means of any mechanical

action (Rinaldi and Capdevila 2006).

(Rinaldi and Capdevila 2006). No energy (e.g. vibration or

shaking) was applied to the soil during the

test. Figure 7b shows some pictures of the loess aggregates retained in various sieves. The distribution

curve corresponding to the destructured specimen was obtained in a similar fashion but the aggregates

in this case were manually broken.

6 ENGINEERING PROPERTIES

6.1 Collapse

As described previously, particle shape, origin and particle gradation result in a poorly accommodated

and open fabric. When water permeates into the soil structure bonding forces between the silt and sand

particles weaken and the structure collapses under moderate external forces, or even under the self

weight of the soil. Figure 8a shows the collapse of loess during compressibility tests. The tests were

carried out under constant vertical stress on one sample at natural moisture content and another that had

been saturated. Several processes take place in the soil as a result of saturation that reduce the shear strength of

binders:

(a) Suction forces gradually decrease as saturation increases.

(b) Double layers hydrate and depending on thickness and available cations in solution, the net balance between attractive and repulsive forces may cause the dispersion of particles (Reginato and Ferrero 1973).

(d) Even low-soluble cements (e.g. carbonates, gypsum, silica amorphous) loose strength as suction is reduced (Alonso and Gens 1994). The acidity of the pore water has a significant effect on cement stability. Highly acidic leachate (e.g. organic acids) dissolves carbonates. It follows from this discussion that strength, stiffness and collapse will be mainly affected by the initial

void ratio and moisture content of the soil. Other factors include: soil fabric, the chemical composition

of the saturating liquid, the amount of soluble salts, the amount of non-soluble cementing agents, and

the depth of burial or the level of external loads. In the past decades, a number of methods and formulas

have been developed to evaluate the potential of collapse for different loess formations around the world. Vertical Stress [kN/m²] 0 2 4 6 8 10 12 0 100 200 300 400 Vertical Stresses [kN/m²] D e p t h [m] Saturated YieldSelf-Collapsible Layer Overburden Pressure (b) 0.00 2.00 4.00 6.00 8.00

10.00

12.00

14.00 10.00 100.00 1000.00

V e

r t i

c a

l s

t r a

i n

[%

] Saturated Natural σ_{ys} σ_{yu} Magnitude of collapse (a)

Figure 8. a) Double oedometer test used to determine the magnitude of collapse in loess; b) Comparison of the

overburden and saturated yielding pressures used to evaluate the thickness and subsidence of a self-collapsible layer

of loess.

Extensive review and discussion of this topic has been presented by Rocca (1985), Rocca et al. (1992)

and Redolfi and Oteo (1994).

Today it is common to use the double oedometer test (Jennings and Knight 1957), the pinhole test

(ASTM D4647), and the slaking test (ASTM D 4644) in the assessment of loess. Figure 8 demonstrates

how the double oedometer test is used in practice to determine collapsibility. The yielding (or collapse)

pressure (σ_y) at any depth are determined in the laboratory from oedometer test performed on soil at

natural water content and saturated (Figure 8a). The saturated yielding pressure (σ_{ys}) is then compared

with the in-situ overburden pressure (σ_o) at the same depth. The magnitude of the potential settlement is

evaluated as,

where, e is void ratio at natural moisture content, e_s and e_u are the vertical strains at saturation and

natural water content respectively, and are determined from the double oedometer test at the same

vertical overburden pressure. The value δ_r is also known as magnitude of collapse. A similar procedure

can be followed to determine the magnitude of settlement under a shallow foundation, with the total

induced vertical pressure used to determine the magnitude of collapse.

Figure 9 shows the average values of yielding stress from 420 boreholes in primary loess. At depths

over 4 m the yielding stress is lower than the overburden pressure. Thus, when primary loess is found

below 4 m, there is a strong possibility of self-subsidence if water penetrates into the soil mass. Typically,

densification of the soil is observed depths of up to 2 m, due to natural cycles of drying and wetting.

6.2 Compressibility

specimens remains higher than the void ratio of the

saturated specimen throughout the stress range

covered in this study, which indicates that the structure is not completely destroyed (Fedà 1994, and 2000).
0 50 100 150 200 250 300 0 100 200 300 400 500 600 V s [m / s]
Undisturbed Destructured 0,00 0,05 0,10 0,15 0,20 0,25 0,30
1 10 100 1000 σ_v [kPa] σ_v [kPa]

ϵ_v Undisturbed Destructured (a) (b)

Figure 11. Load-deformation (a) and shear-wave velocity (b) curves of undisturbed and destructured specimens

of saturated loess subjected to k_o loading. The destructured specimen was prepared at an initial dry unit weight

$\gamma_d = 12.9 \text{ kN/m}^3$. The data for the undisturbed specimen is the same as that of Figure 11. (Data from Rinaldi and

Claria, 1999).

Piezoceramic bender elements were mounted on the top and bottom platens of the oedometer cell

to monitor the evolution of the shear wave velocity (V s) during loading (in a similar fashion to that

described by Brocanelli and Rinaldi 1999). Figure 10b displays the results. The shear-wave velocity in

soils depends on the mean confining pressure (σ_m), the void ratio, the degree of saturation (S), and the

degree of cementation (c) of the soil (Cascente and Santamarina 1997; Cho and Santamarina 2001 and

Fernandez and Santamarina, 2001). Unsaturated specimens exhibit significant shear wave velocity even

at zero confinement, and the value of initial V s decreases as the degree of saturation increases. Therefore,

V s is controlled by suction and cementation at low stress. As load is applied, V s increases until a peak

value is reached near the yielding or collapse pressure. At high stress, the velocity-stress responses for the

unsaturated specimens appear to asymptotically approach the

velocity stress response for the saturated

specimen. This trend suggests that the effect of the applied stress becomes dominant over the effect of

suction and cementation at high stress. The velocity V_s in the saturated specimen increases continuously

with the applied stress, resembling the standard power velocity-stress response in uncemented, dry or

saturated granular media. These results suggest that the power function usually employed for modelling

V_s with respect the confining pressure can not be applied here for the unsaturated specimens as discussed

by Rinaldi and Claria (1999). Furthermore, the function proposed by Leroueil and Hight (2002) which

introduces an additional term to account for the effect of suction can not be also considered here as the

peak velocity corresponding to the collapse of the structure is not modelled.

Figure 11 shows compressibility curves and measured shear-wave velocity for remolded (destructured)

and undisturbed specimens tested at a similar dry unit weight, and both saturated. The compressibility

and shear-wave velocity of the undisturbed specimen are higher than that of the remolded specimen at

the same vertical stress. This behavior clearly reflects the effect of the initial structure of the soil in

the natural condition, as given by cementing binders since suction here is null as the samples are fully

saturated (Rinaldi and Claria 1999, Leroueil and Hight, 2002). Notice that there is no tendency of the

undisturbed compressibility curve to reach the fully destructured curve even at the highest stress levels

used in the test.

The type of fluid used for saturation can have some effect

on the compressibility of soils (see for

example Leroueil and Hight 2003). Figure 12 shows the compressibility response of a compacted loess

specimen when saturated with fluids of different chemical compositions. Note the samples saturated

with sodium and magnesium chloride swelled at the minimum applied stresses . This result is revealing

in that, although the clay content of the loess is about 17%, the hydration of double layers has some

effect at the macroscopic. The low compressibility obtained for the specimen saturated with acetic acid

is attributed here to the observed generation of gas through the reaction of the acid with carbonates

in the soil. Saturation with sodium hydroxide increased the initial stiffness and the yielding pressure,

resembling the behavior of the structured soil. It is considered that the fluid increased the pH, favoring

the generation of stable silicates from the calcium ions and puzzolanitic minerals (volcanic glass) present

in the loess, as described by Quintana and Redolfi (2001).

Zeballos et al. (1999) evaluated the model of Alonso et al. (1990) for primary loess tested in a suction

controlled oedometer cell. Figure 13 describes the variables involved in the elasto-plastic model. The -6 -5 -4 -3 -2 -1 0 1 2 3 1 10 100 1000 Vertical Stress [kPa] V e r t i c a l S t r a i n [%] OHNa (0.5 N) CH3COOH (1 N) MgCl2 (0.5 N) NaCl (0.5 N) Deionized Water

Figure 12. K_o -load-deformation curves of compacted loess at a dry unit weight of 17 kN/m³ and saturated with,

variously, sodium hydroxide, acetic acid, magnesium chloride, sodium chloride and deionized water (Rinaldi et al.

1999). 10 100 1000 Net Pressure (kPa)

S p

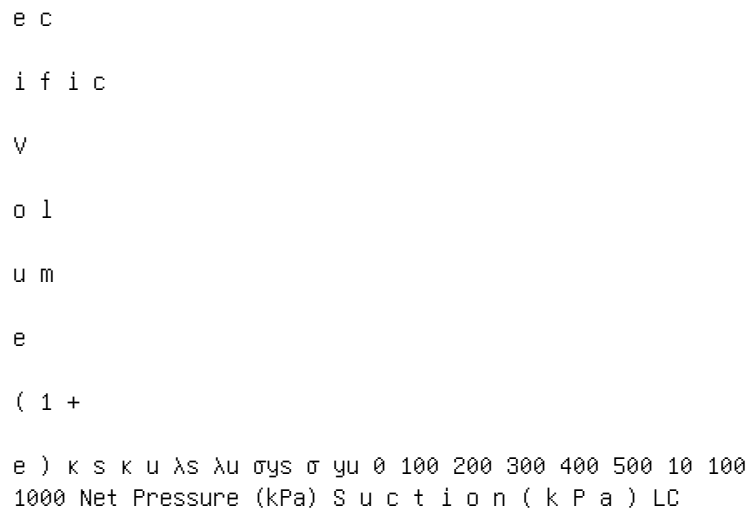


Figure 13. The elasto-plastic model used to model the compressibility of loess.

loading-collapse yielding curve (LC) separates the elastic and plastic regimes. An expression for the LC

line was proposed by Josa et al (1992),

where: σ_{ys} and σ_{yu} are the saturated and unsaturated yielding pressure respectively (refer to Figure 13),

σ_r is a reference pressure, $l = (\sigma_{yu})_{\max} / \sigma_{ys}$, $(\sigma_{yu})_{s=\infty}$ is the yielding pressure at highest suction and α is

an experimental constant. κ_s and λ_s are the slopes of the compressibility curve in the elastic and plastic ranges in saturated

conditions, κ_u and λ_u are the slopes of the compressibility curve at a constant suction value, and κ_w is the

slope of the compressibility curve at constant net pressure in the suction plane. In the model of Alonso

the relationship between λ_u and λ_s is written as:

Table 5. Parameters values obtained by Zeballos et al. (1999) for primary loess tested in a suction controlled cell.

Main physical parameters of the soil are: dry unit weight

11.9-13.1 kN/m³, liquid limit 20.4 to 25.6, and plastic limit

17.8-21.

$(\sigma_{yu})_{s=\infty} = \sigma_{rk} w$

(kPa) (kPa) $\alpha \beta m r k s \lambda s k u \lambda u (p \sim 0)$

430 45 1 1 9.56 0.70 0.002 to 0.035 0.085 to 0.122 0.049
0.137 -0.004 0 50

100

150

200

250

300

350 0 100 200 300 400 500 600 700 σ_v [kPa]

G

m

a

x

[M

P a

] G m a x [M P a] $w\% = 6.42$ $w\% = 15.88$ $w\% = 18.06$
Saturated 0 50 100 150 200 250 300 350 0 100 200 300 400
500 600 700 σ_v [kPa] Undisturbed Destructured

Figure 14. a) Variation of shear modulus G_{max} with vertical pressure for undisturbed specimens of primary loess

of Figure 11, b) Effect of destructuration on G_{max} of saturated loess.

where r is the ratio of the slope of the virgin compression line at the highest suction $(\lambda_u)_{s=\infty}$ to λ_s , and

β is an experimental constant.

In Table 5, the model parameters obtained for primary loess are summarised. From the results it can

be seen that k_u is slightly higher than k_s , resembling that could be attributed here to the effect of suction

on the stiffness of cement binders. The value of k_w determined at close to zero net pressure is negative,

contrary to the expected swelling (positive slope) predicted by the model of Alonso.

6.3 Stiffness

The maximum shear modulus (G_{max}) can be computed from the shear wave velocity (V_s) and the soil

mass density (ρ):

Figure 14a shows the variation of G_{max} with the vertical stress (σ_v) for the data plotted in Figure 10.

Although there is a reduction in the shear wave velocity after the collapse load is applied, the mass

density increases significantly after collapse. As a result there is no matching reduction in G_{max} after

collapse. The saturated specimen exhibits higher values of G_{max} than the unsaturated specimens at high

stresses, because the higher compressibility of the saturated specimen results in a denser medium (refer

to Figure 10). The effect of soil structure on G_{max} is presented on Figure 14b for the saturated specimens

of Figure 11b.

The in-situ G_{max} of loess can be evaluated from the measured shear wave velocity and equation (4).

From results of Rinaldi et al. (1998) obtained from the average of four down-hole tests the following

relationship can be obtained (see Figure 15):

In equation (5), σ_o is the mean confining pressure

expressed in kPa, and the shear-wave velocity

is in m/s. Rinaldi et al. (1998) also studied the correlations of shear wave velocity with blow number
 0 200 400 600 800 1000 1200
 0 50 100 150 200 Mean Confining Stress (kPa)
 Shear Wave Velocity (m/s)
 DH1 DH2 DH3 DH4 Serie5

Figure 15. Variation of shear-wave velocity measured in-situ with mean confining pressure for primary loess in the

city of Cordoba. 0.01 0.1 1 10 100

G

/

G m a x Sample Mr1 (w% 9.9) Sample Mr2 (w% 16.9)
 Sample Mr3 (w% 30.6) Sample Mr4 (w% 46.1) Buy Mud Seed
 e Idriss, (1970), Sands 0.1 0.2 0.3 0.4 0.5 0.6 0.7 0.8 0.9
 1.0 γ [10 4] Isenhower, (1979), San Francisco Stokoe et
 al. (1980), San Francisco Buy Mud Borden et al. (1996),
 Residual soils(MH) Stokoe y Lodde, (1978), San Francisco
 Buy Mud Hardin y Drnevich (1972a), upper and lower bounds
 for all soils

Figure 16. Variation G/G max with shear strain of primary loess compared with other soils reported in the literature

(Claria, 2003).

(N) of the standard penetration test and tip resistance (qc) of the static penetration test and obtained the

following expressions:

where qc is expressed in kPa and the shear wave velocity is in m/s. As could be expected, the degradation curve of elastic shear modulus of loess with shear strain is

highly dependent on water content and confining pressure. Figure 16 shows the modulus degradation
 Deviator Stress, q [kPa] 0 25 50 75 100 125 150 175 200 225
 250 0.0E+00 2.0E-02 4.0E-02 6.0E-02 8.0E-02 Axial Strain, e
 Deviator Stress, q [kPa] 25 50 75 100
 125 150 175 200 225 250 0.0E+00 2.0E-02 4.0E-02 6.0E-02
 8.0E-02 Axial Strain, e a 10 kPa 20 kPa 40 kPa 80 kPa 10
 kPa 20 kPa 40 kPa 80 kPa (b) (d) 0

25

50

75

100

125

150

175

200

225

250

0.0E+00 2.0E-02 4.0E-02 6.0E-02 8.0E-02 Axial Strain, ϵ_a

D

e v

i a

t o

r S

t r e

s s

, q

[k

P a

] 10 kPa 20 kPa 40 kPa 80 kPa 0

25

50

75

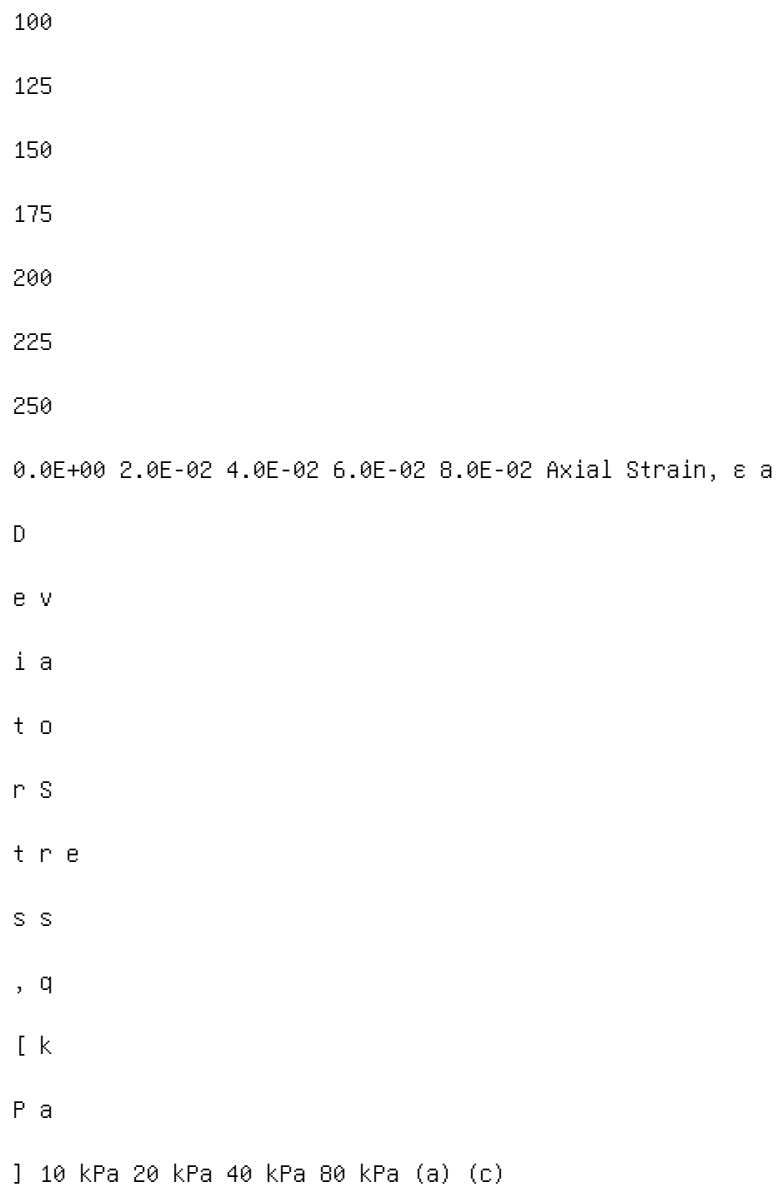


Figure 17. Stress-strain curves of undisturbed and remolded loess specimens ($\gamma_d = 12.5 \text{ kN/m}^3$) tested at different

confining pressure and moisture conditions measured in the triaxial cell using LDT to measure strain. (a) Undisturbed

at natural moisture content; (b) Undisturbed and saturated; (c) Remolded at natural moisture content, and (d) Remolded

and saturated (results from Rinaldi and Capdevila 2006).

curve obtained from resonant column tests of various loess

samples tested at different values of water

content. The normalized modulus values for all samples plot in between the boundaries reported for

most soils. The more highly saturated samples reach larger strain levels than the less saturated samples.

The more highly saturated samples also have a lower normalized modulus, for a given strain, than the

less saturated samples. The linear threshold strain (γ_e) separates the elastic and plastic regimes and is

determined from the degradation curve at the shear strain corresponding to a normalized shear modulus

(G/G_{max}) greater or equal to 0.9 (Santamarina et al., 2001). In figure 16 it can be seen that γ_e decreases

with increasing degree of saturation. Notice that (G/G_{max}) of saturated specimens (MR4) plot close to

the non plastic curve corresponding to sands. As saturation decreases, the curves of (G/G_{max}) approach

to the curves corresponding to a more plastic behavior.

6.4 Shear strength

Figure 17 shows the stress-strain behaviour of drained triaxial compression tests on loess samples,

performed at natural water content and saturated. Measurements of strain were obtained by local dis

placement transducers (LDT), up to 5% strain. The measurement device used was similar to that presented

by Goto et al. (1991). Samples were trimmed from block specimens recovered at 4 m depth from an open

trench excavated in the south of Cordoba city. Preparation and testing are described in Rinaldi and

Capdevila (2006). The test results (Figure 17) show that the unsaturated samples develop much higher

deviatoric stresses than the saturated specimens. In general, it is observed that, at high strain levels (5%),

the higher the confining pressure the higher is the deviator stress. The saturated specimens tested at the

a confining pressure of [20 kPa or less], develop a slight peak stress. All of the samples develop strain

hardening. The higher the confining pressure, the higher is the increment in the rate of hardening. Satu

rated samples show a clear yielding locus, which indicates a brittle behavior. In contrast, the unsaturated, 0 5 10 15 20 25 30 35 40 45 50 0 20 40 60 80 100 120 140 160 180 σ [kPa]

τ

[k P

a] Undisturbed Remolded

Figure 18. Mohr-Coulomb failure envelopes corresponding to the undisturbed and remolded samples tested in

drained (CD) and saturated conditions. Shear strength corresponds to the 5% strain level (data from Figure 17b

and 17d). 0 10 20 30 40 50 60 70 80 90 100 0 50 100 150 200 250 300 350 σ [kPa]

τ

[k P

a] Undisturbed Remolded

Figure 19. Mohr-Coulomb failure envelopes for unsaturated, undisturbed and remolded samples tested in drained

(CD) conditions. The shear strength corresponds to the 5% strain level (data from Figure 17a and 17c).

undisturbed specimens and the remolded specimens exhibit a more ductile behavior and the yielding

locus is not clearly identified. The unsaturated specimens tested at less than 20 kPa developed shear

bands (stress localization) at failure, while the other specimens bulged when loaded to failure. In Figure 18 the

failure envelopes from CD triaxial tests on saturated, undisturbed and remolded

specimens are compared. The samples had a similar dry unit weight (data from Figures 17b and 17d).

The Mohr circles correspond to the deviatoric stress determined at the 5% strain level which is usually

adopted for these type of curves where there is not a distinguishable failure point. The figure clearly shows

the slightly higher shear strength of the undisturbed specimen at low confining pressures. The difference

between the envelopes reduces as the confining pressure increases. The envelop corresponding to the

undisturbed specimen shows a true cohesion intercept at zero confining pressure. Figure 19 displays

the failure envelopes of the undisturbed and remolded specimens tested at natural moisture content at a

similar unit weight (data from Figure 17a and 17c), also for a deviatoric stress determined at 5% strain

level. The remolded specimens show slightly higher strength values than the undisturbed samples, and

both envelopes show a cohesion intercept. The result highlights that in the unsaturated condition, shear

strength is mainly governed by water menisci and the influence of structure and cementation is of little

significance. Figure 20 compares the envelopes corresponding to the shear strength determined at 5%

strain level and at yielding for the saturated samples. The results show that the difference between both

envelops increases with confining pressure. Both envelopes develop a cohesion intercept which can be

interpreted in terms of the slightly cementation of particles. The grain size distribution curve corresponding to the undisturbed (structured) and the fully remolded

(unstructured) samples are shown in Figure 21. The grain

size distribution curves obtained for three

specimens tested in the triaxial cell at natural water content and the same confining pressure of 40 kPa,

but sheared to different strain levels are superimposed in the same figure. This result clearly show that $\phi = 5$

10

15

20

25

30

35

40

45 σ 20 40 60 80 100 120 140 160 180 σ [kPa]

τ

[k P

a] Failure Envelope Collapse Envelope

Figure 20. Mohr-Coulomb failure envelope(s) at 5% strain level and envelope at yielding for undisturbed, saturated

samples tested in drained (CD) conditions. Data from Figure 17b. 0% 10% 20% 30% 40% 50% 60% 70% 80% 90%

100% 0.010.1110100 Diameter [mm]

%

0

f P

a s

a n

t Remolded Soil Tested Soil Conf. 40 kPa $\epsilon = 6\%$ Structured
Soil Tested Soil Conf. 40 kPa $\epsilon = 0.5\%$ Tested Soil Conf. 40

kPa $\epsilon = 2\%$

Figure 21. Grain size distribution curves of undisturbed (structured), fully destructured (remolded) samples of loess

and specimens sheared to different strain levels at the same confining pressure (40 kPa) in the triaxial test, tested in

unsaturated conditions.

the size of the cemented aggregates does not change significantly, even after shearing up to the 6% strain

level reached in these tests. The complete destruction of aggregates shown by the curve corresponding

to the fully remoulded loess seems difficult to achieve at this confining pressure. It is possible that there

would be a greater change in the fabric of the loess if the tests were carried out at a higher confining

pressure.

The stress-strain behavior of loess can be interpreted in terms of the interplay between water content,

strain level, confining pressure and degree of cementation. Before yielding, the stress-strain behavior

corresponds to a cemented material, with a linear relationship between volumetric and axial strain

(Airey and Fahey 1991; Maâtouk et al., 1995; Rinaldi and Capdevila 2006). Before yielding, no plastic

strains are developed, and the stiffness of the loess is controlled, at least initially, by the cemented

bonds. In saturated samples, yielding occurs as a consequence of decementation and the degree of

debonding is controlled by amount and type of the cement, and the confining pressure (Saxena and

Lastrico 1978). The weakly cemented loess requires very low strain levels for yielding (usually less

than 0.5%). Decementation may also takes place by the application of high confining stresses alone. 0 1 2 3 4 5 6 7 8 9 10 60 80 90 100 Degree of Saturation (%) F r i c t i o n A n g l e (°) 0 10 20 30 40 50 60 70 80 90 100 U n d r a i n e d S h e a r S t r e n g t h (k P a) Friction Cu 70

Figure 22. Variation of undrained shear strength parameters of undisturbed loess specimens with respect to the

degree of saturation (modified from Nuñez et al. 1970).

For the samples tested, the required confining pressure to cause decementation was higher than 80 kPa

(see Figure 18). In unsaturated samples, decementation occurs progressively and yielding is mainly

controlled by water meniscus. Thus, the yielding locus takes place at much higher strain levels than that

corresponding to the saturated sample. After yielding, the shear strength may be mainly frictional or cohesive, depending on cement strength,

capillary forces and confining stresses. A high degree of cementation, high suction forces and low

confining pressures produce a blocky structure after yielding (Lambe, 1960) and friction is governed

by the size distribution and shape of the aggregates. In this case, stress localization and shear bands

are developed. When Loess is saturated, the post-yielding strength is frictional, and dependent on

the magnitude of the confining stresses. In unsaturated specimens, decementation is progressive. The

post-yielding strength is initially dominated by cohesion and shows little dependency on the confining

pressure, until a given strain level, where decementation become significant and frictional behavior

becomes dominant. The stress-strain behavior of the remolded specimens at the natural unit weight shows similar behavior

to that of normally consolidated soils. For a given unit weight, the shear strength is mainly determined

by water content and confining pressure. For unsaturated samples, the slightly higher shear strength

envelope relative to undisturbed specimens (see Figure 19) deserves further research. Reasons for the

difference might include the effects of aging and the development of water menisci. Shear strength parameters for loess as determined in undrained triaxial tests (UU) are highly dependent

on the degree of saturation as shown on Figure 22 (Berezantzev et al. 1969; and Nuñez et al. 1970).

Note that the undrained Strength (c_u) at full saturation tends to a typical value of 15 kPa while as it can

be expected, the friction angle tends to zero. Penetration tests results (e.g. SPT and CPT) in loess must

be carefully interpreted (Reginato 1971), as high values of penetration resistance could indicate either a

low degree of saturation or a high degree of cementation.

6.5 Permeability

Figure 23 displays the hydraulic permeability of a natural sample of primary loess as a function of the

hydraulic gradient. The tests were performed in a flexible wall permeameter. Notice that during the

first stage, which involved increasing the hydraulic gradient, the permeability coefficient increased.

The specimen was initially unsaturated, and saturation could not be reached at the low gradient applied

initially. Once saturation was achieved, the permeability coefficient remained approximately constant at

1.5.10⁻⁵ cm/s, irrespective of reductions or increases in the hydraulic gradient. 10 7 10 6 10 5 10 4 0 5 10 15 20 25 Gradient (#) k s [c m / s] First Increasing Gradient Second Increasing Decreasing Gradient

gradient. k 2 .10 6 . n 28.23 10 09 10 08 10 07 10 06 10
5 10 04 10 03 10 02 0.30 0.32 0.34 0.36 0.38 0.40 0.42 0.44
Porosity (n) k s (c m / s) Deionized NaCl c= 2%

and Cuestas 2002).

sensitive to small changes in soil porosity, and that the a concentrated solution of sodium chloride used

The effect of fabric on permeability can be appreciated from Figure 25, which shows how the perme

order of magnitude higher in the dry side of the compaction curve than the permeability obtained on

the wet side of the curve, reflecting the influence of different soil fabrics. Studies performed by Delage

et al. (1996) on silty clays showed that, on the dry side of the compaction curve, the particles formed

aggregates arranged in a porous structure. On the wet side of the compaction curve a clay matrix is

formed around the silt grains, filling the intergranular voids. 14 15 16 17 18 8 16 20 24 γ_d [k N / m ³] 10 7 10 6 10 5 8 16 20 24 Moisture Content w [%] k s [c m / s e c] 12 12

Figure 25. a) Standard Proctor test performed on a sample of loess. b) Permeability tests performed on compacted

samples of loess at constant compaction energy and at different moisture content and density (Rinaldi et al. 1999). Zeballos et al. (2002 and 2005) reported

permeability results obtained for undisturbed and remolded

unsaturated samples of loess. Tests were performed in a one-dimensional flow cell (rigid wall) under

constant hydraulic gradient. Water infiltration was registered until the steady-state flow condition was

achieved. The parameters in the unsaturated state were calculated from fitting of the model of van

Genuchten (1980) to the measurements of Figure 4.

where, θ_r and θ_s are the residual and saturated volumetric water content, k_ψ is the unsaturated hydraulic

conductivity, s is the matric suction, $m = (1 - 1/\eta)$, η , and p are experimental parameters. Table 6 displays the function parameters experimentally determined for compacted and undisturbed

specimens of loess. From the parameters of Table 6 and equations (7) and (8), the unsaturated hydraulic

conductivity can be determined for compacted and undisturbed loess. However, as the hydraulic con

ductivity was not measured under controlled suction, the validity of these Equations could not be

checked.

Table 6. Permeability function parameters for compacted and undisturbed samples of loess (Zeballos et al. 2005).

Variable Unit Natural soil Compacted soil at 100% T-99

Saturated water content, θ_s cm³/cm³ 0.50 0.34

Residual water content, θ_r cm³/cm³ 0.10 0.12

Parameter, p 1/cm 0.0018 0.0009

Parameter, η # 2.41 1.40

Saturated permeability, k_s cm/s 1.2×10^{-5} 1.5×10^{-7}

6.6 Electrical conductivity

The total electrical conductivity of soils is a complex

parameter, but can be expressed as the combination

of the low frequency conductivity (usually known as Ohmic conduction) with conduction losses due to

different polarization mechanisms that arise from the presence of bounded electrical charges (e.g. ions

in double layers, and dipolar molecules of water). In complex notation conductivity is written as (see for

example Rinaldi and Francisca, 1999),

where σ_0 is the ohmic low frequency conductivity, or the conductivity at frequencies much lower than

that corresponding to the relaxation process. $k^* = k' - j k''$ is the frequency dependent complex dielectric

permittivity of the medium, which considers the energy adsorbed and dissipated during the relaxation pro

cess, $k'(\omega)$ and $k''(\omega)$ are the real and imaginary permittivities respectively, ϵ_0 is the vacuum permittivity

$8.85 \cdot 10^{-12}$ F/m, j is the complex number $(-1)^{0.5}$, and ω is the angular frequency.

6.7 Ohmic conductivity

The ohmic conductivity (σ_0) is determined, in practice, from measurement of total soil conduction (equa

tion 9) at frequencies below 1 kHz, such that $(\sigma_s)_{\omega \sim 0} = \sigma_0$. The inverse value of the electrical conductivity

is the electrical resistivity ($\rho = 1/\sigma_0$), which is used in some well known electrical geophysical explo

rations (e.g. vertical electrical sounding and electrical imaging). Rinaldi and Cuestas (2002) presented

a comprehensive study relating the nature of primary loess to the DC electrical conductivity of the soil,

and discussed some engineering applications. In saturated loess the main parameters that govern ohmic

conductivity are the type of pore fluid and the porosity. These can be captured by means of the Archie's law,

where F is the soil formation factor, σ_w and σ_s are the ohmic electrical conductivity of the pore fluid and

the whole soil respectively, n is the soil porosity and a and m are experimental constants. Figure 26 shows

the fitting of Archie's model to experimental data obtained for primary loess. The parameters obtained

from regression analysis that best fitted the results were: $a = 0.66$ and $m = 2.49$ (both dimensionless).

For unsaturated loess Rinaldi and Cuestas (2002) obtained the following relationship:

Here the parameter F_s is the inverse of the resistivity index, and θ_v is the volumetric water content

The exponent p was obtained as:

Figure 27 shows the agreement between equation (11) and experimental results. Notice that volumetric

water content is the main parameter that influences the resistivity index, while the dry unit weight has

only a small effect on the index. Equation (11) can be simplified for a given natural sample of loess and

a specific electrolyte as:

where ξ and the exponent λ are experimental constants. Equation (13) is plotted on Figure 28 together

with the measured electrical conductivity of different specimens of loess at various dry unit weights. $F = 0.66 n^{-2.49}$

Porosity (n)	Formation Factor (F)	Natural Samples	Washed Samples
0.2	0.2	16.2 kN/m ³	13.5 kN/m ³
0.3	0.3	15.9 kN/m ³	15.2 kN/m ³
0.4	0.4		
0.5	0.5		
0.6	0.6		

Figure 26. The formation factor of saturated specimens of primary loess as a function of porosity. θ_v Volumetric Water Content θ_v Resistivity Index $1/F_s$ γ_d 16.2 kN/m³; Washed Samples γ_d 13.5 kN/m³; Naturals γ_d 15.9 kN/m³; Naturals γ_d 15.2 kN/m³; Naturals

Figure 27. Influence of water content and dry unit weight on the resistivity index of primary loess (modified from

Rinaldi and Cuestas, 2002).

6.8 Dielectric permittivity

Dielectric permittivity is the fundamental parameter that governs electromagnetic wave propagation

through materials. Modern geophysical techniques in geotechnical engineering, such as the ground

penetrating radar (GPR) and the time domain reflectometer (TDR), are based on the interpretation of

the spatial variation of dielectric permittivity (Daniels et al. 1988; Annan 1992). The real and imaginary

components of dielectric permittivity of loess are displayed in Figure 29 in the frequency range of 20 Mhz σ s
0,0006 0 v 1,573 0,00 0,05 0,10 0,15 0,20 0,25 0,30 0,35
0,40 0 10 Volumetric Water Content [%] E l e c t r i c a l
C o n d u c t i v i t y [m h o / m] 20 30 40 50 60

Figure 28. Electrical conductivity of various specimens of primary loess as a function of the volumetric water

content. The measurement frequency was 1 kHz and the dry unit weight varied from 11 to 20 kN/m³ . 1 10

100

1000 0 20 40 60 80 Volumetric water content (%)

R

e a

l p

e r

m

i t t

i v i

t y

(k

') 200MHz 1GHz 25MHz (a) 0 20 40 60 80 0.1 1 10 100 1000
 10000 Volumetric water content (%) I m a g i n a r y p e r
 m i t t i v i t y k ' 200MHz 1GHz 25MHz (b)

Figure 29. Real permittivity (a) and Imaginary permittivity (b) of primary loess as a function of frequency and

volumetric water content (Rinaldi and Francisca, 1999).

to 1.3 GHz, for samples of loess prepared at different water contents (θ_v). From these results it can be

observed that at low frequencies, both k' and k'' depend on water content. The larger the amount of

water, the larger are the values of k' and k'' for a given frequency. The observed frequency dispersion

of k' here can be attributed to the Maxwell-Wagner polarization mechanism. Models to consider this

mechanism in the frequency range of this study have been presented in the literature by Arulanandan

and Smith (1973), Wobschall (1977) and Thevanayagam (1995). At frequencies larger than 200 MHz the

dielectric permittivity of soils is mainly dependent on the amount of free and bound water.

Rinaldi and Francisca (1999) discussed the dielectric properties of loess and evaluated different

formulas to account for the observed behavior. From regression analysis, the authors determined a

correlation that best fitted the data for the dielectric constant of loess, measured at 1 GHz, as a function

of the volumetric water content. 0,01 0,10 1,00 10,00
 100,00 0 20 Volumetric water content (%) P e n e t r a t i
 o n (m) 25MHz 1 GHz 40 60 80

Figure 30. Penetration depth of electromagnetic waves in loess as a function of frequency and water content. Equation (14) can be use to evaluate the volumetric water content from the measurement of the real

component of dielectric permittivity (e.g. in the TDR) and

to evaluate the propagation wave velocity (v)

of electromagnetic waves in the loess, assuming low loss media, as:

where $c = 3 \times 10^8$ m/s is the velocity of light in vacuum. When planning a geophysical investigation it is necessary to know the depth to which the electro

magnetic waves can penetrate. For practical applications it can be computed using the following

expression (Annan, 1992): Figure 30 shows the calculated penetration depth of electromagnetic waves in loess at different fre

quencies and water content. From these results it becomes clear that at the range of natural water content

usually found in loess (>20 by volume) the penetration of the waves is less than one meter, which greatly

limits the use of geophysical tools, like the ground penetrating radar, in loess.

7 ENGINEERING PROBLEMS

The oldest Pleistocene loess (secondary or re-worked loess) of the Pampean Formation is a deposit that

has been overconsolidated by a combination of past-pressure and cycles of desiccation. These resulted

in the soil having high stiffness and shear strength. Therefore, the soil is able to support high imposed

loads from structures such as large factories, nuclear power plants, and buildings (Bolognesi 1975 and

Nuñez 1975). The youngest Holocene-primary loess has a high collapse potential, and is prone to erosion as water

comes in contact with the soil. There are a large number of structures founded on these soils that have

experienced large settlements due to collapse of the ground. Erosion has resulted in significant damage

to infrastructure such as railways, canals, and other structures, particularly in special agriculture zones

(see Figure 2). Mechanisms that cause the wetting of loess include the modification of the infiltration-evaporation

balance, a rising water table, leakage of water reservoirs and pipes, and local accumulation of rain-water.

A generalized increase of water content, such as that due to a rising of water level, may lead to a general

land subsidence and uniform settlements. In contrast, local wetting of the soil, such as occurs due to

pipe leakage, may cause significant differential settlements and lead to the complete destruction of the

structure.

Foundations on primary loess can be accomplished by a number of procedures and techniques (Moll

et al. 1988):

a. Improvement of the soil by reducing porosity (e.g. pre-wetting, dynamic compaction, blasting, densi

fication by pile driving, etc.), or by strengthening the soil skeleton (e.g. addition of silicates, heating,

jet grouting, etc.).

b. Minimizing the conditions that favor the collapse of the soil. A number of alternatives can be adopted

to prevent the wetting of the ground, such as: providing an impervious coating to buried water pipes,

the construction of side-walks around the building and drainage systems to reduce the effect of rain.

c. Adopting alternative construction methods that reduce the impact of soil-collapse on the structures

(e.g. the reinforcement of brick walls, design of isostatics structures, etc.).

d. Reducing contact pressure by pre-excavation of the soil. This procedure is not recommended in

self-collapsible loess.

All of the listed solutions have been used in Argentina, but with different degree of success. Pre-wetting

and dynamic compaction provide good results for relatively deformable facilities like embankments for

dams and railways (Moll et al. 1979). Solutions for light-weight structures and small buildings usually

combine the stiffening of the structure and taking measures to minimize the wetting of the soil. Ground

improvement by means of chemical agents or heating yielded very good results for some specific

problems, but were expensive when compared with other methods that were equally effective, such as

deep foundations. The stiffening of structural elements or the use of isostatic structures only reduces

the cracking of the structure, and the risk of structural damage. Pre-wetting by flooding requires large

amounts of water, and can adversely affect the foundations of nearby structures. Therefore, this alternative

is usually only recommended for the construction of dams. This technique was used, in Argentina, for

the construction of the Rio Hondo dam (Moretto et al. 1963).

For deep foundations in collapsible soils, it was previously thought sufficient to place the tip of the pile

at a depth large enough to minimize the risk of wetting of the soil. This solution proved to be inadequate

since the pile increased the weight of the structure, and the collapse of the soil around the pile generated

negative skin friction that resulted in the pile becoming overloaded. The mechanism of load transfer of

piles in collapsible soils has been extensively studied by Redolfi and Oteo Mazo (1992). Some structures

founded on piles have experienced significant damages after collapse of the soil around the pile. Today,

pile foundations of important structures are designed to reach a stable soil layer that would be able to

withstand most of the load transferred by the structure to the pile. Self-collapsible layers should be

identified. The design of the piles should take into consideration the potential for negative skin friction

in such layers.

Finally, compacted loess does not have a collapsible structure, behaves like a non-saturated silty soil,

and exhibits good characteristics of strength and stiffness. Therefore, loess can be used in the construction

of embankments for roads and earth dams. The main limitation in this application is the potential

dispersion characteristics of the clay fraction of the soil that can result in internal and surface erosion.

8 CONCLUSIONS

Loess deposits cover an important area of Argentina. The most recent formation originated from the

aeolian deposition of particles derived from the Andes mountains located to the west side of the country.

Primary loess is characterized by an open structure of fine sand and volcanic silt particles that are

weakly bonded. Soluble salts and other weak cementing agents such as carbonates and silicates are the

main binders. The main difference between Argentinean loess and other loess deposits is the presence

of volcanic glass and sometimes puzzolanic fly ash. As saturation increases, clay buttresses hydrate,

cementing binders weaken, suction vanishes and the structure collapses even under the self weight of

the soil. Thus, capillary forces and cementation cause an increase in stiffness, yielding stress, shear

strength, and tendency to stress localization. The most popular test for evaluating the collapse of loess

is the double oedometer. From this test, the collapse potential and yielding pressures can be evaluated,

both at natural water content and at saturation.

Below the yielding stress (collapse), the soil behaves as a linear elastic material, and the stiffness is

governed by saturation and the degree of cementation could also be the case with partial saturation. At

higher strain levels, the collapse potential of the soil skeleton is governed by a complex interplay between

the applied external pressure and the internal forces due to cementation, matric suction and coulombic

attraction-repulsion forces developed at the level of colloidal particles. The compressibility and elastic modulus of loess are largely influenced by water content (suction)

and the structure of the soil. It has been shown that at the higher vertical stresses used in this study, the

compressibility curve of undisturbed, saturated loess does not reach the fully remolded curve. Similar

evidence is presented here respect to loess tested in triaxial compression. The structure of the soil is

preserved even at large strains. Electrical ohmic conductivity is governed by porosity, type of fluid and electrolyte concentration.

For a given soil formation, with an approximately constant electrolyte concentration, there is a good

correlation between conductivity and volumetric water content. At frequencies in the megahertz range,

the high conductivity of loess reduces the penetration of electromagnetic waves. This makes the use of

ground penetrating radar unsuitable for geophysical investigation in loess. The design of foundations in loess requires special structural and design considerations, to

reduce the

damage caused by soil collapse, or alternatively, the implementation of soil improvement. In the design

of deep foundations, the effect of negative skin friction due to self-collapsible layers must be carefully

considered.

ACKNOWLEDGEMENTS

The funding for the research presented here was provided by the Research Council of Argentina

(CONICET) and Cordoba Research Agency (ACC). All this support is gratefully acknowledged. The

authors also wish to acknowledge the contribution of many students involved in the research and also

the comments and suggestion given by S. Leroueil and N. Shirlaw in the revision of this manuscript.

Special thanks are due to Prof. Carlos Santamarina for the motivation and the encouragement provided

to the writers to present this work.

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Geotechnical characterization and properties of Venice lagoon

heterogeneous silts

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ABSTRACT: To protect the city of Venice against recurrent flooding, a huge project has been under

taken involving the design and construction of movable gates located at the three lagoon inlets for

controlling tidal flow. The design of the gate foundations demands accurate geotechnical characteriza

tion of the Venice lagoon soils, composed of a

predominantly silt fraction combined with clay and/or sand forming an erratic interbedding of various sediments, whose basic mineralogical characteristics vary narrowly. To this end, two typical test sites in the lagoon – the Malamocco Test Site and the Treporti Test Site – were selected for the mechanical soil characterization and for a site-specific calibration of the most widely used geotechnical investigation tools. Besides a presentation of the main geological features of the lagoon sediments, the paper presents and discusses the main outcome of the research carried out at the Malamocco Test Site and some results from the Treporti Test Site.

1 INTRODUCTION

The worldwide-known historic city of Venice continues to preserve a rather precarious equilibrium with the surrounding lagoon, although the margin of security is being eroded at an ever increasing rate. The rate of environmental deterioration is being accelerated by the increasing frequency of the flooding of the historic city – referred as to “acqua alta” (i.e. literally “high water”) – caused by the natural eustatic rise of the sea level, by natural subsidence and by a regional man-induced subsidence, the latter particularly important between 1946 and 1970 (Butterfield et al. 2003). Figure 1 shows the Venice lagoon in the present configuration: through the inlets of Malamocco, Lido and Chioggia flows the tide, becoming, under particular atmospheric conditions, acqua alta, that means flooding of the historic city.

The first comprehensive geotechnical and geological studies on Venice lagoon soils were carried out between 1965 and 1975. Mineralogical and paleontological

studies on sediments from the site Motte

di Volpego (Figure 1) were performed by Bonatti (1968). In 1970 the Italian Government through the

Consiglio Nazionale delle Ricerche (CNR) commissioned a 1000 m deep borehole, located at Tronchetto

(Figure 1) to estimate the aquifer and aquitard system and the geotechnical properties relevant to an

evaluation of the subsidence problem (Favero et al. 1973; Rowe 1973; Ricceri & Butterfield 1974).

Between 1970s and 1980s, the shallowest ground of the lagoon has been extensively studied, but only

through standard geotechnical investigations necessary for the design of the foundations of new large

industrial settlements on the mainland.

Relevant information on the mechanical behavior of lagoon clayey/silty materials sampled at Fusina

(Figure 1), a small village facing to the inner lagoon where a thermoelectric power plant is located, was

also provided by Cola (1994).

To protect the city of Venice and the surrounding lagoon against the recurrent flooding, a huge project

was undertaken at the beginning of the '80s under the directive of Italian Government, involving the

design and construction of movable gates located at the three lagoon inlets (Gentilomo 1997). These

gates, controlling the tidal flow, temporarily separate the lagoon from the sea at the occurrence of

particularly high tides, whose annual frequency is continuously increasing (Harleman et al. 2000).

Therefore, comprehensive geotechnical studies were necessarily carried out to characterize the Vene

tian soils at the inlets to achieve a suitable design of the movable gates foundations. Preliminary standard

Figure 1. View of the Venice lagoon with the locations of test sites.

geotechnical investigations were therefore performed in the late 1980s, to draw relevant soil profiles

along with cross sections of the three lagoon inlets and to estimate significant geotechnical properties

for a preliminary selection of gate foundations. From the geological and geotechnical investigations carried out so far, it turned out that the main

characteristic of the lagoon soils is the presence of a predominant silt fraction, combined with clay and/or

sand. These form a chaotic interbedding of different sediments, whose basic mineralogical characteristics

vary narrowly, as a result of similar geological origins and common depositional environment. This latter

feature, together with the relevant heterogeneity of soil layering, suggested to concentrate the researches

on some selected test sites, considered as representative of typical soil profiles, where relevant in-situ and

laboratory investigations could be carried out for a careful characterization of the Venetian lagoon soils. The first test site, namely the Malamocco Test Site (MTS) (Cola & Simonini 1999, 2002) was located at

the Malamocco inlet (Figure 1). Within a limited area, a series of investigations that included boreholes,

piezocone (CPTU), dilatometer (DMT), self-boring pressuremeter (SBPM) and cross hole tests (CHT)

were performed on contiguous verticals. In addition a continuous borehole was carried out for a very

careful soil mineralogical classification. One of the main aims of the MTS was to evaluate the reliability of the most widely used charts or

correlative equations for the piezocone and dilatometer interpretation on the basis of the comparison with

the results of the laboratory tests, with the aim of characterizing soil profile and the main geotechnical

properties in an economical way (Simonini & Cola 2000; Ricceri et al. 2002). Due to the influence of

high soil heterogeneity, relevant difficulties were however encountered in comparing the relevant soil

properties with those directly estimated from site tests. The absence of any clear horizontal layering

provided no absolute certainty in carrying out a comparison of material properties exactly within the

same type of soil. The comprehensive laboratory test program completed at MTS (Cola & Simonini 2002; Biscontin

et al. 2001, 2006) emphasized the very heterogeneous nature of the Venice soils: in order to define

even the simplest properties with a certain degree of accuracy, a relatively large number of tests was

necessarily required. In addition, due to the high silty content and the low-structured nature, the sediments

are extremely sensitive to stress relief and disturbance due to sampling, thus affecting a careful the stress

history evaluation and a satisfactory stress-strain-time mechanical characterization. A new test site was therefore selected, namely the Treporti Test Site (TTS) (Simonini 2004), which is

located at the inner border of the lagoon, very close to the inlet of Lido (Figure 1). The goal of TTS was

to measure directly in-situ the stress-strain-time properties of the heterogeneous Venetian soils. At the

TTS a vertically-walled circular embankment, loading up the ground to slightly above 100 kPa, was very

recently constructed, measuring, along with and after the construction, the relevant ground displacements

together with the pore pressure evolution. To this end, the ground beneath the embankment was heavily

instrumented using plate extensometer, differential micrometers, GPS, inclinometers, piezometers and

load cells. Boreholes with undisturbed sampling, traditional CPTU (Gottardi & Tonni 2004) and DMT

(Marchetti et al. 2004), seismic SCPTU and SDMT (Mayne & McGillivray 2004) were employed to char

acterize soil profile and estimate the soil properties for comparison with those directly measured in situ.

Besides the presentation of some geological features of the lagoon sediments, the paper collects and

discusses the main results concerning the characterization of the Venice lagoon heterogeneous sandy

and clayey silts based on field and laboratory tests carried out mostly at MTS and TTS.

2 GEOLOGICAL DESCRIPTION OF THE VENICE LAGOON

2.1 Geological and recent history

The Northern Italian lowlands, namely the Padana and Veneta plains, were formed through the fluvial

transport of sediments coming from the erosion of the surrounding Alps and Apennines.

At the end of the Pliocene epoch, the sea level was much higher than today and the Padana and Veneta

plains were submerged.

The Pleistocene epoch was characterized by several glaciation and interglaciation periods with alternat

ing regression and transgression of the shoreline. At the apex of last Würmian (Wisconsinian) glaciation,

the shoreline was located around two hundred kilometres from the present position and, therefore, the

Padana and Veneta plains together with a part of the Northern Adriatic Sea were emerged.

Then, a warmer period set in about 15,000 years ago and the sea level rose during the de-glaciation

period, reaching, between 7000 and 5000 years ago, a value slightly higher than the present one. The

origin of the Venice lagoon is traced around 6000 year ago, during the flandrian transgression, with the

sea water diffusing into a pre-existing lacustrine basin.

In the Venetian lagoon, the upper hundred metres below mean sea level (MSL) are characterized by a

complex system of sands, silts and silty clays chaotically accumulated during the Würmian glaciations

(Favero et al. 1973). The Holocene epoch is responsible only for the shallowest lagoon deposits, up to

10-15 m below ground level.

The top layer of Würmian deposits is composed of a crust of highly overconsolidated very silty clay,

commonly referred as to caranto, on which many historical Venetian buildings are founded through

driven wooden piles. It was subject to a process of overconsolidation as a result of exsiccation during

the 10,000 year emergence of the last Pleistocenic glaciation. Moving from the mainland towards the

shoreline, the caranto layer lies at depths increasing from less than 5 m to about 16 m below MSL

(Gatto & Previatello 1974).

The present lagoon morphology is the consequence of the intensive human action and of the recent

environmental damages. In fact, since the first Venetian citizens settled on the islands (around XII

century), the main engineering activities concerned with the conservation of efficient communications

between lagoon and sea and the preservation of the city insularity (Colombo 1986). More particularly,

to avoid lagoon filling the Venetians diverted the rivers

Brenta, Sile, Piave into extensive canals around

the lagoon periphery and constructed robust stone-walls to protect the channel and island banks against

erosion. The shape of the three lagoon inlets has been also continuously modified: after the second

world war, the sea bottom at the Malamocco inlet was very deeply dredged to allow the large oil tankers

to reach the Venice/Marghera port through the so called "oil canal". All these modifications caused a

continuous decrease of sediment balance, with a significant reduction of the area covered by marshes

and wetlands. Figures 2a,b show a comparison between the lagoon morphology in the XII century with

that of the XX century.

To appreciate the high level of interbedding and interdigitation characterizing the youngest soils up

to 50-60 m of depth, soil profiles from Fusina, Motte di Volpego, Tronchetto, Malamocco and Treporti

are sketched in Figure 3. Note that the majority of soils belong to sands, silts and silty clays with the

presence of several layers of peat: the high heterogeneity is clearly evident at larger scale, but significant

material variation may be observed even at centimetre scale. Mineralogical and paleontological studies

of cores, carried out up to 50-60 m below MSL at Motte di Volpego (Bonatti 1968), Tronchetto (Favero

et al. 1973), Malamocco (Curzi 1995) and, more recently at the Lido inlet (Dip. Scienze della Terra di

Ferrara et al. 2004), detected five depositional units.

The shallowest Holocene unit is mainly composed by marine sands with significant shell content along

with the shoreline and by finer sediments with significant organic matter in the inner part of the lagoon.

The underlying units are mainly formed by fluviatile sediments organized in fining-upward structures,

sometimes eroded or lightly desiccated at the top, due to a temporary regression of the shoreline or to

brief emersion periods. The thin peaty layers should be related to the occasional presence of lacustrine

environment. The rate of accumulation is relatively high, about 3 mm/year in average.

In each profile the caranto is easily recognizable for the typical grey-yellow colour and the presence

of numerous fissures with oxidation traces. The caranto layer, slightly sloping toward the shoreline, has

thickness varying from a few of centimetres up to some metres. Figure 4 (from Gatto & Previatello 1974) (a) (b)

Figure 2. Morphology of the Venice lagoon: 700 years ago (a) and at present (b). Legend

Figure 3. Typical soil profiles of the Venice lagoon.

Figure 4. Map of caranto in the central part of the lagoon (from Gatto and Previatello 1974).

Figure 5. Mineralogical composition for bulk samples, sand fraction and clay fraction of the sediments from

the MTS.

shows a reconstruction of the caranto plan distribution based on data coming from different sites in the

lagoon: to note that the caranto, locally eroded by rivers during the Würmian emergence, covers a great

part of the lagoon area.

2.2 Characteristics of the sediments and porewater pH

To determine the mineralogical composition of Venetian sediments, samples from MTS were analysed

by X-ray diffractometric technique (Curzi 1995).

Particularly three analyses were performed: on the bulk

samples, on the sand fraction and on the clay fraction, the latter being separated through sedimentation in

water and then treated with HCl. Figure 5 depicts the profile of the mineralogical composition obtained

from the three analyses, while Table 1 summaries the overall composition: note that the carbonates,

mainly a mixture of detrital calcite and dolomite crystals, are generally the most abundant component.

Quartz, feldspar, muscovite (2M micas) and chlorite are other significant components.

Table 1. Major components of some samples from MTS (X Ray-diffractometric analysis).

Sample Calcite & Chlorite & Chlorite/ Illite/

depth Dolomite Quartz Feldspar Muscovite Kaolinite Smectite
Smectite

(m) (%) (%) (%) (%) (%) (%) (%)

7.14 37.2 35.7 15.8 6.3 5.0

7.67 27.4 45.0 22.0 3.8 1.8

8.45 44.7 40.5 3.4 5.7 3.7 2.0

9.98 25.3 47.0 20.2 4.5 2.4

10.88 43.6 26.3 14.2 8.6 5.6

11.83 39.5 34.6 15.4 7.6 2.9

12.88 54.1 23.1 16.4 1.7 1.5

19.20 26.5 25.7 43.8 2.0 2.0

21.15 9.5 42.3 31.8 11.2 4.6 0.6

28.08 53.6 28.6 8.9 4.5 4.2

34.88 22.4 33.6 24.6 14.0 5.4

37.82 72.2 16.5 8.7 1.3 1.3

39.25 68.7 21.6 2.4 4.5 2.8

42.90 74.6 17.8 3.1 2.9 1.6

47.60 4.0 43.3 43.0 4.3 2.4

52.14 3.1 50.5 39.3 4.5 2.6

54.61 54.4 29.1 8.9 4.9 2.7

55.80 79.3 16.2 2.9 0.8 0.8

59.51 19.5 66.3 9.9 1.1 2.2 1.0

Figure 6. Electromicroscope photos for granular soils from MTS: (a) sand, (b) silt (Cola and Simonini, 2002). Sandy soils show two distinct types of petrographic sources, namely the granitic or Padana province

and the limestone-dolomite or Veneta province. The first one, characterized by a siliceous-clastic com

position, is due to sediments from the basins of the Po and Adige rivers whereas the second one, showing

prevalently carbonate sediments (with dolomite more abundant than calcite), is shared among the basins

of Brenta, Piave, Livenza and Tagliamento rivers. The Padana and Veneta contributions are mixed

together, the first one prevailing at higher depths (Favero et al. 1973). Sands appear immature, with a mean rounding index, evaluated in accordance with Powers' scale

(Powers 1953), ranging between 0.23 and 0.34. Typical grain shapes of sand and silt are shown in

Figures 6a,b: quartz and feldspars grains are generally angular, suggesting they were not transported for

any extended length of time. Similar conclusions may be drawn by the fact that feldspars and micas are

generally relatively fresh with extensive alteration appearing occasionally on crystal surfaces. Along with the variation of the grain-size distribution from sands to clays, the carbonate and quartz

feldspar fractions decrease when the content in clay

minerals increases. As shown in the third column of Figure 5, the composition of the carbonate-free clay fraction

seems to be rather homogenous at various levels: clay minerals, not exceeding 20% of total weight,

are mainly composed of illite (50-60%) with chlorite, kaolinite and smectites as secondary minerals.

Figure 7. Electromicroscope photos for cohesive soils from Fusina (Cola 1994): (a) sample at 4.4 m of depth;

(b) sample at 12.3 m of depth.

X-ray diffractometer traces indicated that small amounts of quartz and feldspar are also present in the

clay fraction.

Illite, kaolinite and chlorite appear generally highly crystallized, suggesting a detrital origin. Small

amounts of smectites and interstratified illite-smectite minerals, due to chemical alteration occurred in

the depositional environment, are occasionally present.

Figures 7a,b depict electromicroscope photos taken on undisturbed silty clay samples collected at

Fusina at depths of 4.4 m and 12.3 m respectively (Cola 1994). The upper sample of Holocene epoch is

characterized by particle packets connected in a relatively continuous and regular assemblage with an

alveolar structure, probably due to a low energy lagoon environment. In the lower sample, the particles

seem to be aggregated with an irregular structure in small, well-defined packets and arranged by edge

to-face or edge-to-edge contacts, characteristic of a higher energy depositional environment. This latter

feature is in accordance with the observations of Bonatti (1968) who noted, in samples collected at

Motte di Volpego (below 10 m from GL), chaotic structures and micro-cross stratification, these being

characteristic features of sediments deposited from turbulent, rapidly moving masses of water such as rivers.

Comparing in Figure 8a the CaCO_3 profiles determined at Motte di Volpego, Tronchetto, MTS and

TTS, it can be noticed that the carbonates (Calcite and Dolomite) are almost the major component, with

percentages exceeding of 50-70% with the exception of an about 3-5 m thick layer with reduced CaCO_3

concentration, at depth varying between 20 and 27 m below MSL. Some other low-carbonate episodes

exist both at higher and smaller depths, especially at MTS. The reductions of CaCO_3 together with the

presence of peat or organic soils - relatively common in the lagoon area - could be attributed to lacustrine

sedimentation episodes.

Profiles of pH measured at Tronchetto and MTS are depicted in Figure 8b. Slightly lower values were

recorded at MTS, especially at intermediate depths. Oscillation of pH at both sites can be noted, with the

higher values probably related to low CaCO_3 content, being the presence of the carbonatic environment

a neutralizer of water acidity.

2.3 Heterogeneity

In order to check the soil heterogeneity, TAC scanning was carried out on some MTS samples.

Figure 9a shows the density profile in Hounsfield units (HU) obtained using medical TAC equipment

of a typical sample taken at 21.7 m below MSL. To convert the measured HU density into KN/m^3 unit

the correlation:

proposed by Cortellazzo et al. (1995) for some North-Italian soils, was used.

Figure 9b reports the TAC density profile converted in SI units together with the in-situ void ratio e ,

the latter determined at 10 mm-spaced intervals. Both quantities give a relatively irregular trend along

the specimen without showing any particular coincidence between the two profiles and with variations

around 10-15% of the average value. 0 20 40 60 80 100
Carbonate content, % 60 55 50 45 40 35 30 25 20 15 10 5 0 D
e p t h b e l o w M S L (m) Motte di Volpego Tronchetto
Malamocco MTS Treporti TTS 6.5 7 7.5 8 8.5 9 9.5 pH 60 55
50 45 40 35 30 25 20 15 10 5 0 D e p t h b e l o w M S L (m) (a) (b)

Figure 8. (a) Carbonate profiles at Motte di Volpego, Tronchetto, MTS and TTS; (b) pH profiles at Trochetto

and MTS. e 0 γ 16 18 20 22 24 B u l k d e n s i t y γ , k N / m ³ 0 4 8 12 16 Distance, cm γ e 0 1.1 1.0 0.9 0.8 I n s i t u v o i d r a t i o , e 0 Port of Malamocco 21.7 m below m.s.l. (b)

Figure 9. TAC analysis on a sample at 21.7 m of depth below MSL in MTS: (a) photo and density in HU units,

(b) density in geotechnical units and void ratio.

3 THE MALAMOCCO TEST SITE

3.1 Soil profile and basic properties

In order to reconstruct the detailed profile of soil at MTS, continuous coring was performed along two

contiguous verticals using double piston samplers, a standard 10 cm diameter sampler and a larger one

with diameter of 20 cm. Freezing technique was used in sandy layers. Figure 10 shows the basic soil properties as a function of depth. For classification purposes the

Venetian soil classes have been reduced to three: medium to fine sand (SP-SM), silt (ML) and very silty

clay (CL). Features to note are:

- The predominance of silty and sandy fraction can be clearly appreciated: the three soil classes occur approximately in proportions of 35% SM-SP, 20% ML, 40% CL and 5% medium plasticity clays and organic soils (CH, OH and Pt). Percentages of silt exceeding 50% are present in 65% of samples analyzed;

Figure 10. Soil profile, basic properties and stress history at the MTS.

- Sands are relatively uniform but moving towards finer materials the soils become more graded: the

coarser the materials, the lower the coefficient of non-uniformity U , whose range increases with the

decreasing mean particle diameter D_{50} .

- Atterberg limits are characterized by average values of liquid limit $LL = 36 \pm 9\%$ and of plasticity

index $PI = 14 \pm 7\%$. Activity $A = PI / CF$ (CF = clay fraction) is low, the great majority of samples

falling in the range $0.25 < A < 0.50$.

- Significant variations have been observed in saturated unit weight γ_{sat} , even in cores a few centimeters

long (specific gravity $G_s = 2.77 \pm 0.03$);

- Void ratio e_0 lies in the range between 0.7 and 1.0 from 19 m to 36 m below m.s.l.; at greater depths,

it is lower and falls in the range 0.6-0.75 (higher values are due to laminations of organic soils).

3.2 Stress history

Overconsolidation ratio OCR, was evaluated on the basis of the estimated preconsolidation stress σ'_p from

one-dimensional consolidation tests, carried out on CL specimens. Due to the very gradual transition

to the virgin compression regime, the estimate of yielding stress was affected by high uncertainty and

therefore not particularly reliable, excluding a few of

more plastic silty-clay samples. No particular

improvement in estimating σ'_p was unfortunately obtained by other methods (Grozic et al. 2003). A

general decrease of OCR with depth can be observed in Figure 10, with the deeper formations remaining

only slightly OC ($OCR \approx 1.2-3.7$). The higher OCR values close to the top of the ground level are due

to the caranto. 10 20 30 40 50 60 Depth from m.s.w.l., m 10
-4 10 -3 10 -2 10 -1 10 0 10 1 U , D 50 / D 0 , I G S U=D
60 /D 10 D 50 /D 0 I GS =(D 50 /D 0)/U D 0 =1 mm

Figure 11. Grain size index versus depth at MTS.

3.3 Grain size index

Since, at least up to the very silty clay fraction, all the sediments originated from one common basic

material, namely siliceous-calcareous sands, it was attempted to establish some improved relationship

between mechanical properties and the grading characteristics. Venetian sands are relatively uniform but moving towards finer materials the soils become more

graded and the grain-size curve displays a larger range, namely the coarser the materials, the lower

non-uniformity coefficient U , whose range increases with decreasing D 50 . On the basis of the above

observations, it was sought to compare in Figure 11 all the pairs (D 50 , U). Note that, even if both

quantities show oscillations of approximately two orders of magnitude, the profiles of D 50 and U are

generally characterized by opposite trends with depth, namely, when D 50 decreases the U increases and

vice-versa. In order to take into account the coupled and opposite variation of D 50 and U into a single

parameter, a new grain-size index I GS was proposed by Cola & Simonini (2002):

where D_0 is a reference diameter equal to 1 mm. In the laboratory characterization of the Venice soils, it was assumed that some significant time

independent material parameters could be related to I_{GS} , accounting for their grading properties. The

significance of I_{GS} is limited here to the range $8 \cdot 10^{-5} \leq I_{GS} \leq 0.12$ and $CF < 25\%$.

3.4 Mechanical characterization from laboratory tests

An extensive laboratory investigation was carried out on the samples belonging to all the three groups. The

experimental program consisted mainly of one-dimensional compression tests, drained and undrained

triaxial tests (using in some tests a non-contact measurement system) and resonant column tests with

a "fixed-free" apparatus. Shear wave velocity measurements have been performed on some samples

using a bender element system in the triaxial apparatus. As already observed in section 2.3 the main

difficulty in preparing the experimental program was the selection of relatively homogeneous samples

representative of the three classes of soil.

3.4.1 One dimensional compression

Figure 12a report some typical 1-D compression curves of CL, ML and SP-SM samples. The response

has been interpreted in terms of constrained modulus M as a function of vertical effective stress σ'_v

(Janbu 1963),

where C and m are two experimental constants and p'_{ref} a reference stress ($p'_{ref} = 100$ kPa).

Figure 12. Void index (a) and normalized constrained modulus (b) vs normalized vertical stress for some 1-D

compression tests on CL, ML and SP-SM soils. SP-SM ML CL
0.0001 0.001 0.01 0.1 Grain size index, I_{GS} 0 50

100

150

200

E x

p e

r i m

e n

t a

l c

o n

s t a

n t

, $C \pm 30$ $C = 270 + 56 \cdot \log I_{GS}$ $A = 0.93$ (a) SP-SM ML CL
0.0001 0.001 0.01 0.1 Grain size index, I_{GS} 0.2 0.4 0.6
0.8 1.0 Experimental exponent, $m \pm 0.1$ $m = 0.30 - 0.07 \cdot \log I_{GS}$ $A = 0.93$ (b)

Figure 13. Material constants C (a) and m (b) as a function of grain-size index.

Figure 12b shows the trend of M as a function of σ'_v / p'_{ref} for the three classes of soil. The lowest values

correspond to the CL class whereas the highest correspond to the SM-SP class.

The experimental constants C and m were related to the grain size index I_{GS} through the following

equations:

Figures 13a,b show the trend of C and m as a function of I_{GS} .

An attempt to describe the one-dimensional compression of Venetian soils using a unified constitutive

model was recently proposed by Biscontin et al. (2006).

3.4.2 Critical state

Typical results of TX-CK $\emptyset$ U tests for the extreme classes SM-SP and CL are shown on Figure 14a in terms

of deviatoric stress q vs. axial strain ϵ_a and pore pressure u vs. ϵ_a . Figure 14b sketches the corresponding

stress-paths in the $p' - q$ plane ($p' =$ mean effective stress).

The evolution of u indicates the presence of both contractant and dilatant phases: the Phase Transfor

mation Point (Tatsuoka & Ishihara 1974; Ishihara et al. 1975) marks the end of the contractant phase,

after which the sample exhibited dilatant behavior. 200 400 600 800 Deviatoric stress, q (kPa) $\emptyset$ 10 20 30 Axial strain, ϵ_a (%) -200 $\emptyset$ 200 Excess pore pressure, u (kPa) SP-SM (0.21, 1.96) CL (0.007, 19.1) CL(0.003, 7.3) CL(0.011, 26.2) D 50 and U inside parenthesis (a) SP-SM (0.21, 1.96) CL (0.007, 19.1) CL(0.011, 26.2) CL(0.003, 7.3) $\emptyset$ 200 400 600 800 Deviatoric stress, q (kPa) $\emptyset$ 200 400 600 Mean effective stress, p' (kPa) SP-SM (0.21, 1.96) CL (0.007, 19.1) CL (0.003, 7.3) CL (0.011, 26.2) D 50 and U inside parenthesis PTP $\phi'_{PTP} = 36^\circ$ $\phi'_{PTP} = 31^\circ$ (b)

Figure 14. Typical undrained triaxial behavior for SP-SM and CL soils: (a) q and u versus axial strain, (b) $p' - q$

stress paths. SP-SM (D) SP-SM (U) ML (D) CL (D) CL (U) 0.0001 0.001 0.01 0.1 1 Grain-size index, I_{GS} 30 32 34 36 38 40 Critical state angle, ϕ'_c ($^\circ$) $\pm 2^\circ$ $\phi'_c = 38.0 + 1.55 \log I_{GS}$

Figure 15. Critical state angle as a function of grain-size index. Non-dilative response was observed only in a few of the tests performed on samples with higher clay

content, but this behavior cannot be considered as representative of typically Venetian soils. The critical

state angle ϕ'_c is reported in Figure 15 as a function of I_{GS} , distinguishing among the three soil classes

and between drained (D) and undrained (U) tests. As

expected, higher values are due to sands (34° - 39°)

whereas silty clays show lower values (30° - 34°).
Critical angle of silts covers a larger range, reaching

also the upper values of sands. It is interesting to notice
the good correspondence between ϕ'_{PTP} and ϕ'_c ,

Figure 16. Maximum dilatancy versus mean effective stress
at failure in triaxial tests.

with scatter from the average value not exceeding $\pm 2^{\circ}$,
without any particular distinction among the

three classes of soil and type of test.

The parameters e_{ref} = void ratio at p'_{ref} and λ_c =
slope of the critical state line in the plane e - $\log p'$

were also estimated and two regression lines, one for
SM-SP-ML the other for CL, determined: the

pair (e_{ref} , λ_c) turned out to be equal to 0.918 and
0.066 for SM-SP-ML and 0.818 and 0.107 for CL,

respectively (Cola & Simonini 2002). The trends, evaluated
in the range between $8 \cdot 10^{-5} \leq I_{GS} \leq 0.12$,

are characterized by the following expressions:

It is interesting to point out that, before using the above
correlations between critical state parameters

and I_{GS} , attempts have been made to relate the same
parameters singularly to D_{50} and U . The results

indicated that ϕ'_c , λ_c , e_{ref} are a function of both
quantities but with opposite effects: for example, ϕ'_c

increases with D_{50} and decreases with U .

3.4.3 Peak strength and state parameter

Figure 16 shows the maximum dilation rate ($-de_v/de_a$)
max measured in drained triaxial compression

against the mean effective stress at failure p'_f . The
same figure displays also the data of a few samples

showing contractant response: in these cases the dilatancy

values were set to zero and the points were

drawn on the $p' - f$ axis.

Features to note are:

- A general decrease of $(-de_v / de_a)_{max}$ with $p' - f$ is observed for both sandy and silty soils. The shaded

area includes the majority of data from tests on sands and silts, while the few data outside this area

are due to silts with high clay fraction (CF is labelled above the dots);

- No particular distinction is evident between the sand and silt behavior;

- Dilatancy vanishes at stress levels beyond about 1 MPa in all samples;

- The very silty clays are characterized by very low dilatancy with no particular trend related to $p' - f$.

Using the Bolton's expressions (1986, 1987):

where the parameter I_R is:

with $D_R = (e_{max} - e) / (e_{max} - e_{min})$ the relative density and Q an experimental constant (Bolton suggested

$Q = 10$ for quartz and feldspar sands and 8 for limestone), the limits of the shaded area are marked using

the values of $I_R = 0.7 \cdot [(9 - \ln p' - f) - 1]$ and $I_R = 0.5 \cdot [(9 - \ln p' - f) - 1]$. This implies that the measured

dilatancy should correspond to a relative density D_R in the range 0.5-0.7 with the parameter Q equal to 9. According to Bolton the peak angle $\phi' - p$ is similarly related to parameter I_R by:

that may be adapted to the Venice soils as follows:

The value 3.9, used here instead of 3 suggested by Bolton, may be partially justified by considering

that the Venetian sands and silts are composed of angular grains, thus providing a higher degree of

interlocking, whereas the data reported by Bolton are mostly referred to sands characterized by a lower

degree of angularity (e.g. Lee & Seed 1967). The state parameter ψ (Wroth & Bassett 1965; Been & Jefferies 1985; Been et al. 1991) defined

here as:

was also calculated on the basis of the void ratio e_{ref} corresponding to the reference pressure

$p'_{ref} = 100$ kPa on the experimental critical state line. According to Wood et al. (1994) the state parameter

can be related by the expression:

where experimental value of β for sands and silts results equal to 40, as shown in Figure 17.

3.4.4 Very small strain behavior

The maximum stiffness G_{max} was determined on specimens from the three classes of soils, using reso

nant column tests (RC) and shear wave velocity measurements performed in triaxial tests using bender

elements (BE). The experimental data were interpreted as function of p' , OCR and e (Hardin & Black

1969, Hardin & Drnevich 1972). For sands, low-plasticity and slightly OC cohesive soils, the effect

Figure 17. Peak angle in triaxial compression as a function of the state parameter.

of OCR is considered negligible (Hardin & Drnevich 1972) and G_{max} can be related to e and p' by the

expression:

where D is a material constant and n is an exponent.

Figure 18 depicts the ratio $G_{max} / (f(e)p'_{ref})$ as a function of normalized mean stress p' / p'_{ref} , distinguish

ing among the soil types and the test types. An exponent n equal to 0.60 could be reasonably assumed

as representative of the trend for all data.

As shown in Figure 19, the dependence of the material constant D on I_{GS} was assessed and fitted by

the equation:

Note that I_{GS} is once more capable of expressing the dependence of G_{max} on the three classes of soils.

3.4.5 Stiffness at small-intermediate strains

Trends of secant shear stiffness determined in undrained compression (C) and extension (E) tests for

some CL samples are displayed vs. shear strain ϵ_s in Figure 20a (Cola & Simonini 1999). The tests were

performed in a special triaxial cell equipped with internal non-contact displacement sensors. The undis

turbed CL specimens were reconsolidated at the in-situ K_0 stress state before increasing the deviatoric

stress up to failure. $0.1 \leq 10$ Normalized mean stress, p'/p'_{ref} 100 1000 Corrected maximum stiffness, $(G_{max}/p'_{ref})/f(e)$ SP-SM (BE) ML (BE) ML (RC) CL (BE) CL (RC) $n = 0.68$ SP-SM (BE) $n = 0.66$ ML (BE) $n = 0.61$ CL (BE) $n = 0.60$ CL (RC) $n = 0.56$ ML (RC)

Figure 18. Corrected maximum stiffness as function of mean effective stress. $0.0001 \leq 0.001 \leq 0.01 \leq 0.1$ Grain size index, I_{GS} 150 200 250 300 350 400 450 500 Experimental constant, D SP-SM (BE) ML (BE) ML (RC) CL (BE) CL (RC) $D = 470 + 26.3 \cdot \log I_{GS} \pm 50$

Figure 19. Material constant D as a function of grain-size index.

Figure 20. (a) Shear modulus vs. shear strain and (b) comparison between secant and reloading modulus in undrained

compression and extension triaxial tests. A general decrease of shear stiffness is observed, with lower values for compressions above $\epsilon_s \approx 0.02\%$.

The limit of linear elastic behavior seems to be in the

range of strains between $\approx 0.003\%$ and $\approx 0.006\%$,

for compression and extension respectively. Considering the plasticity index, the overconsolidation of

the Venetian silts and strain rate applied in the laboratory tests, the above limits of linear behavior appear

to be in agreement with those obtained by other researchers (e.g. Dobry & Vucetic 1987, Tatsuoka &

Shibuya 1992; Leroueil & Hight 2002). Small unload-reload cycles were applied in some tests during the shearing stage in order to check

the effects of straining on reversible behavior. The results are plotted in Figure 20b (specimens VE5-C

and VE5-E) in terms of secant shear stiffness G load and initial stiffness in the reloading phase G_{reload}

against shear strain ϵ_s . Within limits of up to $\epsilon_s = 1\%$, to which corresponds relatively high stress levels

$\eta/\eta_{\text{max}} = 0.92$ and $\eta/\eta_{\text{max}} = 0.68$ in compression and extension respectively, the reloading modulus shows

a slight decreasing trend. Little damage of soil structure should therefore develop up to a shear strain

of 1%.

3.4.6 Anisotropy in elastic range

The stress-strain relation of a cross-anisotropic elastic material given by Graham & Houlsby (1983) is:

where J is a coupling modulus and asterisks have been added to K and G to show that the material is no

longer isotropic. Considering that the volumetric strain increment $\delta \epsilon_v$ is zero in undrained compression tests, it is

possible to write:

If $J = 0$, i.e. isotropic elastic soil, the ratio between pore pressure and deviatoric stress increments must

be equal to 1/3. For materials stiffer horizontally than vertically, the value of J is negative and the pore

pressure ratio $\bar{u}/\bar{q}$ should therefore exceed 1/3. Figure 21 reports $\bar{u}/\bar{q}$ as a function of axial strain for samples VE5-C and VE8-C, already mentioned

in section 3.4.5, together with the estimated limits of pre-yield behavior (Cola & Simonini 1999). Within

these limits the soil seems to be characterized by a small anisotropic behavior with stiffer response in a

vertical direction. $1E-3 \quad 0.01 \quad 0.1 \quad 1 \quad 0.0 \quad 0.1 \quad 0.2 \quad 0.3 \quad 0.4 \quad 0.5$
 $0.6 \quad 0.7 \quad 0.8 \quad \Delta u / \Delta q \quad \varepsilon_a, \% \quad \text{VE5-C} \quad \text{VE8-C} \quad \text{Linear range for}$
 $\text{VE5-C} \quad \text{Linear range for VE8-C} \quad \Delta u / \Delta q = 0.33 \quad (\text{elastic}$
 $\text{isotropic behavior})$

Figure 21. $\bar{u}/\bar{q}$ ratio vs. ε_a in undrained compression. $10^2 \quad 10^3 \quad 10^4 \quad \sigma'_v, \text{ kPa}$

0.000

0.004

0.008

0.012

0.016

C_α

e CH CL ML SP-SM Max σ'_v oedometer tests at MTS (a) 10^{-2}
 $10^{-1} \quad 10^0 \quad C_\alpha \quad 10^{-4} \quad 10^{-3} \quad 10^{-2} \quad C_\alpha$ oedometer tests at
 $\text{MTS} \quad \text{CH-OH} \quad \text{CL} \quad \text{ML} \quad \text{SP-SM} \quad C_\alpha / C_c = 0.015 \quad (b) \quad C_\alpha / C_c =$
 0.050

Figure 22. (a) Secondary compression index vs. vertical stress and (b) vs. compression index from oedometer tests.

3.4.7 Consolidation and secondary compression

Primary consolidation coefficient of CL-ML samples is rather high and in the range $10^{-7} - 10^{-5} \text{ m}^2/\text{sec}$,

thus confirming that Venetian silts are relatively free draining materials.

Secondary compression coefficient $C_{\alpha e} = \frac{de}{d \log t}$ estimated

from the oedometric tests is reported

as function of vertical stress in Figure 22a. Although the tests are carried out on all types of soils, $C_{\alpha e}$

does not exhibit great variation being the index more affected by the stress history rather than from the

grain-size composition.

According to Mesri & Godlewski (1977) and Mesri et al. (1995), the ratio $C_{\alpha e} / C_c$ (C_c is the local slope

of the $e-\log \sigma' - v$ curve) is not dependent on the stress level or material density. Mesri et al. suggested a

small dependence from soil composition, $C_{\alpha e} / C_c$ varying from 0.02 to 0.05 moving from granular soils

to silts and to organic clays, but for the Venetian soils this ratio does not appear particularly dependent

on grain-size composition: from Figure 22b it appears that above ratio of the three classes CL, ML and

SP-SM fall approximately in the same range. The best fitting provides $C_{\alpha e} / C_c = 0.028$ with negligible

differences between the classes CL, ML and SP-SM. A higher value equal to 0.031 is obtained for the

few CH-OH soils.

3.5 Characterization from site testing: piezocone and dilatometer applicability

One of the aims of the MTS was to evaluate the applicability of CPTU and DMT for extensive soil

characterization at the three inlets (and, eventually to provide a site-specific calibration), thus avoiding

the use of more expensive borehole and laboratory testing as well as the scale effects inherent in these

highly heterogeneous soils. The CPTUs at the MTS were carried out using the standard Dutch cone provided by the pore pressure

transducer located along the shaft just behind the cone

(Torstensson 1975, 1977) with a ratio a between

net and total area equal to 0.83. The following symbols have been used in the interpretation of the CPTU:

q_c = cone resistance;

u_2 = pore pressure measured at the cylindrical extension of the cone;

f_s = measured sleeve friction;

$q_t = q_c + (1 - a)u_2$ = corrected cone resistance;

$q_n = q_t - \sigma_{vo}$ = net corrected cone resistance (σ_{vo} = total overburden stress);

$u_2 = u_2 - u_o$ = excess pore pressure, where u_o is water pore pressure in a hydrostatic condition;

$R_f = 100 f_s / (q_t - \sigma_{vo})$ = normalized sleeve friction ratio;

$B_q = u_2 / (q_t - \sigma_{vo})$ = pore pressure ratio. The standard DMT test procedure (Marchetti 1975, 1980) was utilized at the MTS, recording the

gas pressure p_0 to lift the membrane off the sensing device located just beneath the membrane and

the gas pressure p_1 to cause 1 mm deflection. From these two pressure readings Marchetti derived the

three index parameters, namely the material index $I_D = (p_1 - p_0) / (p_0 - u_0)$, the horizontal-stress index

$K_D = (p_0 - u_0) / \sigma'_{vo}$ and the dilatometric modulus $E_D = 34.7(p_1 - p_0)$ where σ'_{vo} is the in-situ vertical

effective stress, and subsequently a series of empirically derived properties through correlations based

on the above three index parameters (e.g. Totani et al. 1999). The results of two typical tests SCPTU19 and MDMT2, carried out at closer distance, are shown in

Figure 23 in terms of the profiles of q_t , f_s , u_2 , p_0 , p_1 and V_s , where the latter is the shear wave velocity

measured at various depths by the SCPTU and by the CHT.

Figure 23. Profiles of q_t , f_s , u_2 , p_0 , p_1 and V_s at MTS.

Sharp variations of q_t , f_s and u_2 are observed; more particularly u_2 remains below the hydrostatic level

not just in the caranto but also in some cases in the sandy layers. This effect has been observed in the

special circumstances of dense cohesionless soil that exhibits dilation when sheared (Wroth 1984). B_q

rarely exceeds 0.4 at MTS, whereas in other sites higher values have been observed.

The p_0 and p_1 profiles are characterized by the continuous and coupled oscillations of the two pressures,

thus confirming the high degree of heterogeneity already shown by SCPTU19.

In CHT an increasing trend of V_s with depth, from 150 m/s up to 300 m/s at 50 m below m.s.l., is

noted with very small variations among the different soil formations. Some flapping of V_s , measured

with SCPTU at depths from 29 to 34 m and 45 to 52 m, may be due to the presence of some thin peaty

layers, which could influence the propagation of V_s across horizontal soil layering.

On the basis of laboratory test results, an evaluation of the applicability of CPTU and DMT to

estimate some relevant geotechnical properties is considered and discussed in the following sections.

Other information can be found in Ricceri et al. (2002).

3.5.1 Soil classification

Soil classification with CPTU can be obtained by using q_t and f_s alone (Olsen & Farr 1986; Robertson

1990) or with u_2 (Senneset et al. 1982; Robertson 1990).

All the classification methods are based on empirical

charts, with the most widely accepted being

Robertson's where the soils are grouped into 9 classes, accounting also for OCR. Figure 24 reports the

CPTU data superimposed onto Robertson's chart (in terms of q_t vs. R_f and in terms of q_t vs. B_q), dividing

the soils into the four classes, SM-SP, ML and CL and organic soils. The proposed subdivision seems

relatively suitable for the classification of the Venetian soils, which mostly belong to the groups 4, 5

and 6.

The material index I_D was originally used as approximate parameter for identifying the soil type, i.e.

$I_D \leq 0.6$ for clay, $0.6 < I_D < 1.8$ for silt and $I_D \geq 1.8$ for sand. However, it was later recommended to

combine the material index I_D with E_D for better soil classification.

Figure 25 summarizes the data obtained from the MTS and plotted on the improved chart proposed

by Marchetti & Crapps (1981). The two extreme classes of soil, namely the silty clay (CL) and silty sand

(SM-SP) appear to be better defined than those of silts (ML).

Marchetti & Crapps soil classification chart provides indications about the unit weight of soil. Com

paring the γ predicted with that measured in the laboratory, the range of maximum scatter around the

average values does not exceed 15%. Nevertheless, the chart tends to overestimate γ in CL soils while

underestimating that of SM-SP class. $0.1 \leq R_f \leq 100 f_s / (q_t - \sigma_{vo})$

10

100

1000

(q t

-

σ

v o

) / σ

' v

o SP-SM ML CL CH and organic soil 1 2 3 4 5 6 7 8 9 NC OCR
Sensitivity -0.2 0.0 0.2 0.4 0.6 0.8 1.0 1.2 1.4 B q = Δu
2 / (q t - σ v o) SP-SM ML CL CH and organic soil 7 6 5 4
3 2 1. Sensitive, fine grained 2. Organics soils, peats 3.
Clays - Clay to silty clay 4. Silt mixtures- Clayey silt
to silty clay 5. Sand mixtures - Silty sand to sandy silt
6. Sands - Clean sand to silty sand 7. Gravelly sand to
sand 8. Very stiff sand to clayey sand 9. Very stiff, fine
grained (a) (b)

Figure 24. Classification with the Robertson's charts. OL
CL ML SP-SM I D 1 10 100 E D , M P a 0.33 0.6 0.8 1.2 1.8
3.3 S I L T Y C L A Y C L A Y E Y S I L T S I L T S A N D Y
S I L T S I L T Y S A N D S A N D CLAY Marchetti & Crapps
(1981) ($\gamma = 18 . 0 \text{ k N / m }^3$) (17) (16) (16) (19) (2
0) (21 . 5) (17) (18) (18) (17) (19) (2
0 . 5) (21) (19 . 5)

Figure 25. Classification by means of Marchetti and Crapps
chart. -0.1 0.0 0.1 0.2 0.3 0.4 0.5 B q 1 10 0 C R OCR =
 $12.9 \cdot e^{-6.11 \cdot Bq}$ r 2 = 0.55

Figure 26. Relationship between overconsolidation and pore
pressure ratios.

3.5.2 Overconsolidation ratio

The level of u 2 generated during cone penetration, and
consequently the value of B q , are functions of the

type of soil and of OCR. In order to verify if a possible
relationship between OCR and B q exists, the OCR's values
determined

from 1-D compression tests on CL samples were plotted
versus B q in Figure 26. A general decrease

of OCR along with an increase of B_q can be observed, but no reliable correlation between these two

quantities can be established, the level of u_2 being much more dependent on soil type than on OCR. Another attempt to link OCR with CPTU results could be performed as suggested by Mayne (1991),

who proposed a relationship between OCR and the ratio $(q_t - u_2)/\sigma'_{vo}$ on the basis of the coupled use of

cavity expansion theory and of the MCC model:

Figure 27. Overconsolidation ratio versus $(q_t - u_2)/\sigma'_{vo}$ (a) and versus horizontal stress index (b).

where $3 \sin \phi' c / (6 - \sin \phi' c)$ is the slope of critical state line in the (p', q) plane.

The data reported in Figure 27a confirm the relation between OCR and the ratio $(q_t - u_2)/\sigma'_{vo}$. The

curve given by Mayne's theoretical approach for a friction angle of 30° , characteristic of the Venetian

clayey silts, represents approximately an upper limit of OCRs.

Figure 27a also reports the best regression fit obtained on experimental data, with the expression:

which could be used to predict tentative profiles of OCR in the slightly over consolidated range.

The estimation of OCR for clays was proposed by Marchetti relating K_D to OCR from oedometer tests

with the following correlation:

The use of correlation (17) is restricted to materials with $I_D < 1.2$, free of cementation which have

experienced simple one-dimensional stress histories.

Among the improved relationships available in literature, that proposed by Lacasse & Lunne (1988):

takes into account a large range of soil plasticity in the exponent, that varies from 1.35 for plastic clays

up to 1.67 for low plasticity materials.

The applicability of correlations (17) and (18) to MTS data was verified in Figure 27b. The Marchetti

correlation seems to provide a rather good estimation of OCR at the lowest values of K_D . Large differences

compared to laboratory results occur at higher horizontal stress indexes ($K_D > 10$), that is, for the highly

OC caranto at shallow depths. No improvement of data relating to any of the K_D range was obtained by

adopting the Lacasse & Lunne relationship.

If we apply the power regression to all our data independently of I_D and stress history we have the

following relationship:

which could provide, as similar to the piezocone, an OCR prediction in the range $1.7 < K_D < 70$.

3.5.3 Friction angle

The relationship due to Durgunoglu & Mitchell (1975) and modified by Robertson & Campanella (1983)

is based on the correlation between ϕ' , q_t and σ'_{vo} :

ϕ'_{peak} ,

ϕ'_{crit}

($^\circ$) ϕ'_{peak} , ϕ'_{crit} ($^\circ$) ML SP-SM ML SP-SM Eq. (20) Robertson & Campanella (1983) $20 \ 40 \ 60 \ 80 \ \tan \phi'_{peak} = 0.38 + 0.27 \log(q_t / \sigma'_{vo})$ $r^2 = 0.50 \ 1.5^\circ$ (a) $1 \ 10 \ 2 \ 4 \ 6 \ 8 \ K_D \ 32 \ 36 \ 40 \ 44$ ML SP-SM ML SP-SM Eq. (22a) Totani et al. (1999) Eq. (22b) Totani et al. (1999) (b) $\phi'_{peak} \ \phi'_{crit} \ \phi'_{peak} \ \phi'_{crit}$

Figure 28. Friction angles vs. the normalized cone resistance (a) and vs. the horizontal-stress index (b) in sands and

silty sands.

A similar relationship was used in our case and the regression fitting:

is proposed in Figure 28a, where the shaded area defines the range of possible excursion between the

upper and lower values of the peak angle ϕ'_{p} . Observing a linear correlation between K_D and q_t , Marchetti adapted the relationship (20), thus

presenting a new chart that provides ϕ'_{p} of sand from K_D . Recently Marchetti (from Totani et al. 1999)

suggested two direct empirical correlations, which could be used to give lower and upper bounds of the

range of possible friction angles: A comparison between laboratory values of peak and critical friction angles from triaxial tests on

SP-SM and ML soils and the value predicted by Equations (22a, b) is plotted in Figure 28b: the fitting

is rather limited showing a poor dependence of friction angle on K_D .

3.5.4 Undrained strength

From q_t the undrained shear strength c_u can be determined by applying the bearing capacity equation to

the cone resistance. Hence, we have:

where N_k is the cone bearing capacity factor. Several theoretical approaches are available for N_k (Yu &

Mitchell 1998), the latter being theoretically characterized by a large range of variation, between 11 and

19 for NC clays, approaching 25 for OC ones. In the Venetian silty clays the process of penetration is partially drained and therefore the interpre

tation of cone resistance in terms of undrained strength is rather questionable (e.g. Randolph 2004).

Nevertheless, on the basis of c_u measured in the laboratory, tentative values of N_k were calculated for CL

formations. The result is shown in Figure 29a. The scatter

around the average value of 8.5 is relatively

large; equation (23) can be therefore used only for a possible rough estimate of c_u . Considering the dependence of c_u / σ'_{vo} on OCR, Marchetti proposed a correlation between c_u and K_D :

Correlation (24) was recommended only for material with $I_D < 1.2$ and simple loading history. $N_{k,ave} = 18.5$ $r^2 = 0.70$
 $0.1 \leq q_n \leq 6$ MPa

0.0

0.1

0.2

0.3

0.4

c_u

,

M

$P_a N_k = 11$ $N_k = 25$ (a) $1 \leq 10 \leq 100$ $K_D 0.1 \leq 1 \leq 10$ c_u / σ'_{vo}
 o Eq. (24) Marchetti (1980) $c_u / \sigma'_{vo} = 0.17 K_D 0.96$ $r^2 = 0.79$ (b)

Figure 29. Undrained shear strength from triaxial tests vs. net cone resistance (a) and shear strength ratio versus

horizontal stress index (b). $1 \leq q_t \leq 100$ MPa $10 \leq 100$ G m a x /
 $(1 + B q) 4.59$, M P a CHT CL CHT SP-SM-ML
 SCPT CL SCPT SP-SM-ML BET CL BET SP-SM-ML RCT
 CL RCT SP-SM-ML $G_{max} = 21.5 q_t 0.79 (1 + B q) 4.59$ $r^2 = 0.63$

Figure 30. Relationship between maximum stiffness and CPTU parameters.

Undrained strength from triaxial tests is plotted against K_D in Figure 29b: a general increase of c_u

as a function of K_D is appreciated. It may therefore be suggested that the original correlation could be

utilized for a preliminary estimate of c_u for all the

range of K_D .

3.5.5 Maximum shear stiffness

Correlations were proposed between CPT resistance and G_{max} for a large variety of soils (Baldi et al.

1989; Mayne & Rix 1993; Hegazy & Mayne 1995). The major criticism to all these correlations is that

G_{max} is a parameter determined at very small shear strain levels whereas q_t is a quantity measured at

large deformations involving yielding and failure of the soil surrounding the cone. However, as pointed

out by Mayne & Rix (1993), for a given soil, both quantities show similar dependence on the same

parameters, namely the mean effective stress level and void ratio.

The reconstruction of the e profile is a difficult task. The advantage of using the CPTU is given by the

continuous profile of B_q which, in these soils, is mainly a function of pore size distribution rather than

OCR. Therefore, the B_q profile may act as a simple substitute of e profile, in order to take into account

soil type and its structural condition.

To analyze the dependence of G_{max} on q_t and B_q the data from the SCPT, CHT and from laboratory

measurements performed with BE and RC (see section 3.4.4) were used (Simonini & Cola 2000). The

data on Figure 30 show that an association between G_{max} , q_t and B_q is reasonably possible for Venetian

soils. The following multiple regression function applied on data from all types of soils was selected: $0.11 I_D 0.510 15 20 25 R G = G_0 / E_D RCT + BET CHT SCPT R G = 6.71 - 14.2 \cdot \log I_D r^2 = 0.68$

Figure 31. Relationship between R_G and material index. The very small-strain modulus can also be estimated from E_D . The ratio:

is, in some cases, expressed as a function of K_D or relative density (Jamiołkowski et al. 1988), but these

relationships cannot be applied to all soils. Hryciw (1990) expressed G_{max} as a more general function of

K_0 , γ_{DMT} and σ'_{vo} , in which K_0 is given by the original Marchetti (1980) relationship:

and γ_{DMT} is the total unit weight determined from DMT data by means of Marchetti & Crapps' chart. As presented above the errors in estimating γ_{DMT} and K_0 for Venetian soil are not negligible. Con

sequently, we preferred to propose a direct relationship between R_G and I_D : the best fitting was found

in the form of the logarithmic relationship between R_G and I_D , as shown in Figure 31. Using G_{max} data

measured both in-situ and in the laboratory, the relationship can be written as:

4 THE TREPORTI TEST SITE

In order to measure in-situ the stress-strain-time properties and verify the above discussed applicability

of SCPTU and DMT to characterize the Venetian soils, a comprehensive research program was recently

launched. It was concerned with the construction of a full-scale earth-reinforced cylindrical embankment over a

typical soil profile in the Venice lagoon and with the measurement of relevant displacements of ground

together with pore pressure evolution. The project has been developed in the following steps:

- (a) selection of the typical test site in the lagoon area;
- (b) execution of site investigations;
- (c) installation of the monitoring system;
- (d) construction of the embankment;
- (e) monitoring the displacements and pore pressure during

the embankment realization;

(f) execution of laboratory investigations;

(g) repetition of some in-situ tests after the completion of the embankment;

(h) prosecution of monitoring for almost four additional years after embankment completion.

(i) staged bank removal with monitoring of ground displacements in unloading condition. The test site is located just outside Treporti, an old fishing village near the Cavallino shoreline, facing

the North-Eastern lagoon (TTS in Figure 1). Before construction After construction SCPTU SDMT EMBANKMENT H=6.7 m CPT DMT DMT CPTU 14 0 m 5 10 15 20 N 18 16 12 19 17 20 15 11 6 3 q=106.5 kPa z w =0.5 m 13 Borehole S3 S1 S2 SN R = 2 0 m

Figure 32. Embankment plan with location of site investigation tests.

4.1 Site and laboratory investigations

Figure 32 sketches the location of site investigations consisting of 10 CPTU with dissipation tests

and 2 mechanical CPT (Gottardi & Tonni 2004); 10 DMT with dissipation tests (Marchetti et al. 2004);

6 seismic SCPTU/SDMT (Mayne & Mc Gillivray 2004); 4 boreholes. Undisturbed sampling was carried

in the most cohesive formations out by using the classical Osterberg sampler. Two additional CPTU and

four DMT have been also performed after completion of the embankment.

On the Osterberg undisturbed samples standard laboratory tests have been carried out. Plastic liner

samples were used for a detailed classification of soil profile.

4.2 Monitoring instrumentation

The instrumentation installed at TTS was designed to keep

the following main quantities under control:

- surface vertical displacements using 7 settlement plates and 12 bench marks, the latter located outside

the embankment area;

- surface vertical displacement by one GPS antenna, located in the centre of the embankment area and

fixed to the central settlement plate;

- vertical deep displacements by means of 8 borehole rod extensometers;

- vertical strains, along with four verticals, using 4 special multiple micrometers;

- horizontal displacements by means of 3 inclinometers;

- pore water pressure in fine-grained soils by means of 5 Casagrande as well as 10 vibrating wire

piezometers;

- total vertical stress beneath the loading embankment by means of 5 load cells.

Figures 33 and 34 show the instrumentation position and a schematic soil profile, which is described

in the next sections. EMBANKMENT 0 m 5 10 15 20 N
Settlement plate Sliding micrometer Load cell Inclinometer
Casagrande piezometer Vibrating-wire piezometer Rod
extensometer Settlement plate and GPS receiver Bench mark
N S R = 1 5 m R=20 m

Figure 33. Embankment plan with location of monitoring devices. 0 5 10 15 20 m piezometers sliding micrometer settlement plate rod extensometer inclinometer bench mark GPS receiver 0 5 10 15 20 25 30 35 40 EMBANKMENT silty sand sandy silt Alternate layers of sandy silt and peat clayey silt silty clay clay SECTION N-S

Figure 34. Cross-section of embankment with soil profile and monitoring devices.

Figure 35. Trial embankment on completion.

To measure the vertical displacements throughout the

foundation ground very precisely, multiple

micrometers, capable of measuring vertical displacements at 1 m intervals with an adequate degree of

accuracy of 0.03 mm/m (Kovari & Amstad 1982), were selected.

4.3 Embankment construction

Three-meter long prefabricated vertical drains were first installed to speed up the drainage of a shallow

clay layer (about 1.5 thick) and to prevent possible lateral soil spreading during construction.

The cylindrical sand bank construction started in September, 12th 2002 and ended in March, 10th

2003. The bank is formed by 13 geogrid-reinforced sand layers with 0.5 m thickness. The fill was a

medium-fine uniform sand with $D_{50} = 0.14$ mm. As reinforcement a stiff polypropylene geogrid was

used; to avoid any sand flow through the holes of the grid, additional sheets of geotextile were placed

onto the grids before placing the sand. The fill was dynamically compacted to give an average dry unit

weight equal to 15.6 kN/m³.

After completion, the embankment was covered by an impermeable membrane on which 0.2 m of

medium-fine gravel was posed, thus reaching a final height of 6.7 m. Figure 35 shows the bank on

completion.

4.4 Results

The interpretation of test data is still in progress and only some results are shown here. Staged unloading

with bank removal has been planned for the end of 2006/beginning of 2007.

4.4.1 Soil profile, basic properties and stress history

The approximate ground sequence, whose first 40 meters are shown on Figure 34, basically consists of:

(a) 1-3 m: soft silty clay, whose drainage has been improved by prefabricated drains. This thin layer con

tributes significantly to the total settlement of the embankment, even though it cannot be considered

as representative of the Venice lagoon cohesive formations;

(b) 3-8 m: medium-fine silty sand;

(c) 8-20 m: clayey silt with a sand lamination between 15 and 18 m, covering in plan a sector of 270 °

from North to East. This layer constitutes the fine-grained deposit that gave rise to most part of the

vertical displacements;

(d) 20-22 m: medium-fine silty sand;

(e) 22-45 m: alternate layers of clayey and sandy silt;

(f) 45-55 m: medium-fine silty sand.

Frequent laminations of peat are present below 25 m, in layers E and F.

Figure 36 reports, as a function of depth, the basic soil properties determined on laboratory samples.

The profile atTTS is similar to that at MTS with the exception of the stiff OC caranto, which was probably

here eroded during the last glaciation. This confirms the validity of the selected site to reproduce a typical

Venice lagoon ground profile.

Figure 36. Soil profile, basic properties and stress history at the TTS. Additional features to note are:

- From the soil grading reconstruction, the various types of soil occur up to 60 m, approximately, in the proportion: SM-SP 22%, ML 32%, CL 37% and CH-Pt 9%.

- Upper and deeper sands are relatively uniform; finer materials are more graded, the coarser the materials, the

lower the coefficient U , with I_{GS} lying in the same range observed at MTS.

- Except for the organic soils, Atterberg limits of cohesive fraction are similar to those determined at MTS;
- The unit weight γ_{sat} , showing large oscillations with depth, is somewhat lower than that measured at MTS, void ratio e_0 is slightly higher and lying approximately in the range between 0.8-1.1, with higher values due to laminations of organic material. The last column of Figure 36 sketches the profile of OCR, estimated with Casagrande method on the

base of oedometric test results on CL samples as well as from the interpretation of in-situ stress-strain

behavior, as described in section 4.2.4. Note that, according with in-situ response, the soil appears to be

slightly overconsolidated, with a regular trend decreasing with depth. OCR estimated from oedometer

tests shows a major scatter even also in specimens collected at quite the same depth, but this fact may be

due to sampling disturbance and to the difficulties in applying the Casagrande method in silty materials.

Other comments about OCR will be reported in section 4.2.4. Figure 37 reports the compression and recompression coefficients C_c and C_r , the coefficient of

secondary compression $C_{\alpha} = \frac{de}{d \log(t)}$ as well as the coefficients of consolidation c_v and c_h estimated

from laboratory and in-situ tests. The relevant variation with depth of the coefficient of consolidation, 0 10 20 30 40 50 60 Depth below GL (m) 0 2 4 6 8 10 C_{α}, e 10 -3 0.01 0.1 1 10 c_v , c_h (cm²/s) 0 0.2 0.4 C_c , C_r c_h DMT c_h CPTU c_v oedom. C_c C_r .

Figure 37. Profiles of compression indexes and consolidation coefficients at TTS.

characterized by higher c_h with respect to c_v , proves once more the relevant soil heterogeneity. To note

that c_h varies in the same range measured at MTS.

The results of two typical tests CPTU14 and DMT14, carried out near the centre of the embankment

and at closer distance, are shown in Figure 38 in terms of profiles of significant quantities q_t , f_s , u_2 , p_0 ,

p_1 and V_s , where the latter is the shear wave velocity measured by SCPTU and SDMT.

Relevant oscillations of u_2 are clearly evident, thus suggesting large differences in drainage conditions

provided by a continuous layering of different grain size composition in the whole deposit. Pore pressure

parameter B_q lies in the same range measured at MTS, with some occasional values over 0.5. Similarly

to MTS, V_s increases with depth from about 150 m/s to 300 m/s at higher depths.

4.4.2 Pore pressure evolution during construction

The presence of prefabricated drains in the upper silty clay layer A) and the relatively high soil drainage

of all the deeper layers suggested that primary consolidation should have been quite rapid and con

temporary with the embankment construction. In other words, no important delayed deformation due

to consolidation should have been observed, considering that the rate of load increase, required by the

earth-reinforcement construction technique, was low compared to the drainage conditions of the deposit.

This hypothesis seemed to be confirmed by the electric piezometer readings which gave no detectable

pore pressure increase in any layer: the variation of pore pressure in the piezometers appeared to be mostly

controlled by the daily oscillations of the sea tide in the channel facing the embankment area rather than

by the increasing load. Casagrande piezometers measured a maximum water excursion of around 0.4 m

during the whole construction interval. Figure 39 shows the results of the electric piezometer readings

measured in the last loading phase together with the tide excursion measured in the canal.

4.4.3 Vertical and horizontal displacements

Figure 40 shows the evolution with time of the maximum ground settlement measured under the centre

of the embankment. The total settlement on embankment completion (around 180 days) was 381 mm.

Until the last measurement (October, 13th 2005), an additional secondary settlement at constant load of

124 mm was measured, thus giving a total settlement of 505 mm.

Figure 41 depicts the deformed ground surface at the completion of the bank and at the time of the

last measurement, together with the horizontal displacements measured with one of three inclinometers.

Comparing the vertical and horizontal displacements, it appeared that the total vertical displacement

Figure 38. Comparison among the profiles of q_t , f_s , u_2 , p_0 , p_1 and V_s at TTS. -1.0 0.0 1.0 Canal water level (m) -4 -2 0 2 4 Pore pressure (kPa) 917 - 14m 913 - 19m 916 - 22m n° piezometer - Depth 914 - 5m 915 - 10m 4, 0 4, 5 5, 0 5, 5 6, 0 6, 5 0 2 4 6 Embankment height (m) 12/12/02 12/26/02 1/9/03 1/23/03 2/6/03 2/20/03 3/6/03

Figure 39. Pore pressure evolution during the last construction sequence.

is one order of magnitude greater than the maximum horizontal displacement, that is, the deformation

process developed prevalently in the vertical direction. After bank completion, a small horizontal recover seems to have probably occurred, but due to the

low precision of inclinometer used for these readings this outcome cannot definitely be confirmed. The

performance of GPS is also superimposed on the same plot:
 note the excellent agreement between 500 400 300 200 100 0
 Settlement (mm) 0 200 400 600 800 1000 1200 2.5
 Embankment height (m) 6.7 end of
 construction time(days) (q=106.5 kPa) 382 surveying
 measurements GPS measurements

Figure 40. Evolution with time of maximum ground settlement
 under the embankment. 0 5 10 15 Distance from embankment
 axis (m) -500 -450 -400 -350 -300 -250 -200 -150 -100 - 5 0
 Ground surface settlement (mm) 0
 -20 20 40 60 Radial displacement (mm) 30 25 20 15 10 5 0 0
 Depth below GL (m) Bank completed Mar-03 Last
 measurement Oct-05 Inclinator n°3 Sec. SW-NE

Figure 41. Vertical (centre) and horizontal (side) ground
 displacements.

the two methods of settlement measurement. GPS may be
 therefore an attractive alternative for a

continuous vertical displacement monitoring.

It is interesting to analyze the distribution of vertical
 displacement with depth provided by the multiple

extensometers. These measurements are given in Figures 42,
 both in differential and accumulated form

(for the extensometer installed close to the centre). 0 100
 200 300 400 500 Accumulated displacements (mm) Sept 27,
 2002 (H=1m) Nov 11, 2002 (H=2.5m) Jan 10, 2003 (H=4.5m) Mar
 14, 2003 (H=6.7m) Jun 6, 2003 (H=6.7m) Dec 3, 2003 (H=6.7m)
 Oct 13, 2005 (H=6.7m) 0 10 20 30 40 50 Local vertical
 displacement (mm/m) 40 35 30 25 20 15 10 5 0

D

e p

t h

b

e l

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G

L

(

m

) A) Soft silty clay B) MF Silty sand C) Clayey silt with sand lamination D) MF Silty sand E) Alternate clayey and sandy silt

Figure 42. Local and total vertical displacements measured close to the bank centre. The relevant contribution of the upper silty clay A and, particularly, of the silt layer C is clearly evident.

The influence of embankment reduces with depth and beyond 35 m the displacements are very small

and not appreciable with the type of instruments installed at TTS. The differential displacement representation allows for an appreciation of strains, which are mostly

concentrated, excluding the upper layer A, in the silty formation C. The thick continuous line marks the

bank completion. Note that the maximum vertical deformation ϵ_z does not exceed 4% in all the layers

B, C, D and E. The vertical strain ϵ_z plotted as a function of increasing stress applied to the ground (the latter

approximately estimated using an elastic finite element analysis using a realistic distribution of stiffness

throughout the soil profile) allowed for a tentative evaluation of preconsolidation stress. Figure 43 depicts a few of the typical vertical stress σ'_z vs. vertical strain ϵ_z responses (plotted starting

from geostatic vertical effective stress σ'_{vo}) for some 1-m thick layers, recorded using the multiple

extensometer located close to the bank centre. Note the variation of curvature throughout the loading

process, characterized by a much stiffer response at the beginning of the loading phase. Hypothesizing that no delayed deformation due to consolidation occurred along with the loading

phase, the sharp variation of curvature was interpreted in terms of yielding stress σ'_{y-site} . Since strains in

the ground developed prevalently in the vertical direction, it was tentatively assumed that the classical

preconsolidation stress $\sigma'_p \approx \sigma'_{y-site}$. Therefore, OCR calculated as $\sigma'_{y-site} / \sigma'_{vo}$ has been plotted in Figure 44. This estimate of OCR is

particularly relevant, since it is the first attempt to measure directly in-situ the yielding stress of Venetian

materials. In addition, to better understand the origin of overconsolidation at TTS, that is erosion and/or

structuration and/or delayed consolidation, the approach proposed by Perret et al. (1995) and discussed

by Locat et al. (2003) was considered. The estimate of the overconsolidation difference $OCD = \sigma'_p - \sigma'_{vo}$

and overconsolidation gradient $OCG = \sigma'_p / \sigma'_{vo}$ were performed using the only available continuous

OCR profile, that is, that provided by site monitoring and plotted in Figure 44. If $\sigma'_p = \sigma'_{vo}$, OCD is equal

to zero and both OCR and OCD are equal to unity. Following Locat et al., a constant profile of OCD

and OCG with depth means that overconsolidation is due to erosion whereas variation of one of the two

quantities may be due to cementation and/or secondary compression. The profiles plotted in Figure 44 would tentatively suggest that the small overconsolidation ratio of

layers C, D and E should be mainly due to an erosion process occurred during the Pleistocene epoch

Vertical deformation, ϵ_z (%)	σ'_v (kPa)	Depth below GL (m)	OCD	OCG	SAND	SILT	CLAY	Composition (%)
1.6	8.5-9.5	8.5-9.5	-2	0.5	20	40	40	
1.2	10.6-11.6	10.6-11.6	0	1.0	30	30	40	
0.8	11.6-12.6	11.6-12.6	2	1.5	25	25	50	
0.4	13.7-14.7	13.7-14.7	4	2.0	20	20	60	
0.0	15.7-16.8	15.7-16.8	6	2.5	15	15	70	

Figure 43. Vertical strain of the most compressive strata, developed during construction.

Figure 44. OCR, OCD and OCG at Treporti Test Site.

rather than to cementation or secondary compression. This aspect will be further discussed in the next

section.

4.4.4 Stiffness/compressibility from site monitoring

The high quality of measurements provided by the multiple extensometer allowed us to measure the

in-situ soil stiffness, thus avoiding both the scale effect and the stress relief and/or destructuration due

to sampling, particularly affecting the mechanical response of these soils.

The compression behavior observed in the multiple extensometer located close to the bank centerline

(Figure 33), whose stress-strain curves have been shown for a few of layers in Figure 43, has been

interpreted in terms of:

- Initial stiffness M_{i-site} , evaluated as slope of the tangent to the vertical stress σ'_z vs. vertical strain ϵ_z

curves;

Figure 45. Comparison between initial stiffness and maximum stiffness from SCPT test.

- Tangent stiffness $M_{site} = \Delta \sigma'_z / \Delta \epsilon_z$ evaluated for each loading step;

- Compression index C_{R-site} defined as slope of $\epsilon_z - \log \sigma'_z$ curves beyond estimated yielding stress σ'_{y-site} ;

- Secondary compression index $C_{\alpha s-site}$ defined as slope of the $\epsilon_z - \log(\text{time})$ curves after the end of embankment construction. The initial stiffness M_{i-site} representing the maximum site stiffness, is plotted against depth in Figure 45.

For sake of comparison, the trend of maximum stiffness, evaluated with the down-hole seismic piezo

cone and converted into elastic modulus (for elastic isotropic medium) in both confined (M SCPTU) and

unconfined conditions (E SCPTU) using a Poisson ratio = 0.20, is also depicted in Figure 46. As expected,

M i-site is characterized by values lower than the SCPTU stiffness profile, except a few of shallower layers

where M i-site seems to approach the SCPTU stiffness. Normalized modulus (M / σ'_{z}) site of layers B, C and D is plotted in Figure 46 against normalized

current vertical stress ($\sigma'_{z} / \sigma'_{y}$) site induced by the increasing embankment load. Note the sharp variation

of (M / σ'_{z}) site trends for clayey silts of layer C before and after the yield stress, that clearly separates the

behavior in OC from NC range. Both Holocene and Pleistocene sands are clearly characterized by higher

values, whose variation is less influenced by the overcoming of the in-situ yield stress. Assuming tentatively that the deformation process under the bank centerline approaches the 1-D con

dition, the (M / σ'_{z}) site values of the thick silty layer B is compared in Figure 47 with the laboratory values

of the normalized constrained modulus (M / σ'_{z}) lab obtained from one-dimensional compression tests. In

order to compare data as homogeneous as possible, (M / σ'_{z}) lab was calculated through the interpretation

of the recompression curve in an unloading-reloading cycle followed by the virgin compression part

(dashed line shown in the upper-right part of Figure 47). With this procedure, the normalizing yielding

stress is the maximum vertical stress before an unloading-reloading cycle, that is, the precompression

effect on the response in 1-D tests is due only to mechanical imposed precompression, thus reducing

possible intrinsic influence of ageing or secondary compression or structuration on soil stiffness. It is important to notice that field data cross the laboratory trend: the stiffness before yielding stress

is significantly higher compared to laboratory data whereas beyond $(\sigma' z / \sigma' y)$ lab it reduces to significantly

lower values. This may be tentatively explained considering the effect of disturbance and stress relief

due to sampling that removes from laboratory material response any influence of ageing/secondary

compression/structuration. From the above considerations, it seems that the small precompression

observed in Pleistocene formations at TTS should be probably due to the combined effect of mechanical

precompression and ageing, not excluding the possible influence of a very small structuration. 0.5 0.75 1 1.25 1.5 Normalized vertical stress, $(\sigma' z / \sigma' y)$ site 1 10 100 1000 10000 Normalized modulus, $(M / \sigma' z)$ site B) Holocene medium-fine silty sand SM-SP C) Pleistocene clayey silt ML D) Pleistocene medium-fine silty sand SM-SP Holocene sandy sediments

Figure 46. Normalized tangent stiffness vs. normalized vertical stress.

Figure 47. Site and laboratory normalized tangent stiffness vs. normalized vertical stress.

The slope of the site compression curves, whose typical trend is depicted in Figure 43, was interpreted

(after overcoming the yield stress) in terms of virgin compression coefficient CR site and plotted in

Figure 48 as a function of depth. Values of CR lab, defined like the ratio $\frac{e z}{\log(\sigma' z)}$ after yielding in

1-D compression tests, are superimposed on the same graph. Note that CR lab was estimated, as commonly

done in current engineering practice, in a range of vertical effective stress well above that induced in

situ by the embankment loading. 30 20 10 0 Depth bel

o w G L (m) 0 0.1 0.2 0.3 0.4 CR CR lab CR site

Figure 48. Site and laboratory virgin compression coefficients. 10 100 1000 Time (days) 5.0 4.0 3.0 2.0 1.0 0.0 V e r t i c a l s t r a i n ϵ_z (%) Depth (m) 8.5-9.5 m 10.6-11.6 m 11.6-12.6 m 13.7-14.7 m 15.7-16.8 m 17.8-18.8 m Bank on completion

Figure 49. Typical trends of vertical strain vs. time measured with the multiple extensometer. Figure 49 shows typical ϵ_z vs. $\log(t)$ curves evaluated for each 1-m thick layer. The strain-time

trend is characterized by a S-type shape with the final part, corresponding to deformation occurring

after bank construction, fitted in the ϵ_z - $\log t$ plane by a straight line whose slope is assumed to be

the site secondary compression coefficient $C_{\alpha\epsilon\text{-site}} = \frac{\epsilon_z}{\log t}$. A comparison with laboratory $C_{\alpha\epsilon\text{-lab}}$

is tentatively proposed for the layers B, C and D, assuming, again, that the site deformation process

under bank centreline approaches the 1 D response. The comparison is carried out in Figure 50 in terms

of Mesri's approach considering the ratio between primary and secondary compression ($C_{\alpha\epsilon} / CR$) site . As

found at MTS, upper ($C_{\alpha\epsilon} / CR$) site values are due to SM-SP material, intermediate to CL and lower to ML.

However no appreciable difference is noticed among the three different soil classes. ($C_{\alpha\epsilon} / CR$) site ratios 10 -2 10 -1 10 0 10 -4 10 -3 10 -2 $C_{\alpha\epsilon\text{lab}}$ and $C_{\alpha\epsilon\text{site}}$ Laboratory SP-SM ML CL Site SP-SM ML CL $C_{\alpha\epsilon} / CR = 0 . 0 1 5 C_{\alpha\epsilon} / CR = 0 . 0 5 0$ CR lab and CR site

Figure 50. Site and laboratory primary and secondary compression coefficients.

are similar to those from the laboratory, but with upper values due to the deep formation E, (composed

prevalently of silty clay and clayey silt with some important peat lamination) and lower values for the

upper formation C (composed of sandy silts). ($C_{\alpha\epsilon} / CR$)

site for sandy layer of formation B is somewhat external to the typical range suggested by Mesri.

5 CONCLUSIONS

The paper outlined some relevant results of the research carried out so far to characterize the mechanical

behavior of Venice lagoon heterogeneous silts.

The information obtained from the investigation carried out at the Malamocco Test Site is now being

improved by a new experimental study at the Treporti Test Site, whose results, based on the directly

in-situ measurement of the stress-strain-time soil properties, appear to represent a unique and valuable

tool for a suitable calibration of CPTU and DMT and for the formulation of a reliable geotechnical model

aimed at an appropriate design of the mobile gate foundations.

ACKNOWLEDGEMENTS

The authors wish to extend special thanks to: the Consorzio Venezia Nuova, the Magistrato alle Acque,

the research groups of the University of Bologna and of L'Aquila; Prof. P.W. Mayne of GeorgiaTech,

USA; everybody at the geotechnical group of the University of Padova.

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Colombian volcanic ash soils

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ABSTRACT: Volcanic ash soils result from the in situ weathering of volcanic ashes. These soils

exhibit a distinctive behavior that is a consequence of its formation history, current mineralogy and

structure. In this study, we analyze the formation history of Colombian volcanic ash soils to explain

salient characteristics and develop an experimental program to explore thermal, electrical, mechanical

and chemical properties in view of engineering needs. The mineralogical composition of volcanic ash

soils is characterized by the presence of clay minerals that are uncommon in soils of sedimentary origin

such as allophane, imogolite and halloysite. Dissolution and reprecipitation into new minerals has been

accompanied by a large increase in porosity and the development of intercrystal bonding. High porosity

and inherent cementation determine the strength, compressibility and conduction properties of these

soils. In particular, Colombian volcanic ash soils exhibit a dramatic change in stress-strain response

upon remolding. Consequently, the interpretation of field observations and laboratory test results, as

well as the selection of construction operations must be carefully considered within the context of the

formation history of Colombian volcanic ash soils.

1 INTRODUCTION

Volcanic ash soils formed from the weathering of materials ejected during volcanic eruptions. They are

found in regions with intense volcanic activity such as the Pacific Ring of Fire. Approximately 60% of

the soils in tropical countries are volcanic ash soils (Quantin, 1986; Shoji et al., 1993).

Volcanic eruptions have occurred during the last 20,000 years in the Andes Range in Colombia

(Pleistocene and Holocene - Quaternary period). The location of major volcanoes and the correlated

distribution of volcanic ash soils are shown in Figure 1. In the central region, they coincide with the

Coffee Zone and the departments of Antioquia, Caldas, Risaralda and Quindio. In the southwestern

region, they are found in the departments of Tolima, Huila, Cauca, Valle del Cauca and Nariño. And in

the eastern areas, volcanic ash soils are distributed in a random pattern (Bogota and Llanos Orientales).

These deposits play an important role in agriculture, especially for upland crop production.

2 ENGINEERING GEOLOGY

2.1 Source of material - "Volcanic Ash"

Volcanic eruptions start with the ejection of pyroclasts. The cloud of pyroclasts is formed by volcanic

ashes which are made of solidified magma or crust fragments that range in diameter from 0.2 μm up to

2 mm. The primary mechanisms in volcanic ash formation include magma vesiculation (i.e. gases expand

rapidly and generate voids called vesicles), magma fragmentation by very high thermal or mechanical

stresses generated by the interaction with near-surface water, and comminution or pulverization of lava

or crust materials from vent walls (Wohletz & Krinsley, 1982; Büttner et al., 1999).

The energy liberated during the eruption determines the size and elevation reached by the dust cloud.

Volcanic ashes can raise several kilometers into the atmosphere and can be transported hundreds of kilometers. The spatial distribution of volcanoes (■) and volcanic ash soils (shaded area) in Colombia (modified from IGAC, 1995). About 12% of the national territory is covered with volcanic ash soils.

Figure 1. Spatial distribution of volcanoes (■) and volcanic ash soils (shaded area) in Colombia (modified from

IGAC, 1995). About 12% of the national territory is covered with volcanic ash soils.

kilometers away from the source by strong wind currents. Some ashes can remain suspended for days

or months in the atmosphere. Particle shape (sphericity and roughness), size and specific surface,

chemical composition and electrostatic charge determine travel distance away from the source and

settling velocities (Riley et al., 2003). Volcanic ash particles have an initial blocky or vesicular shape. Blocky particles have planar surfaces

resulting from the brittle breaking of frozen lava. Vesicular ashes have drop-like shape. Friction and

abrasion during transport tend to reduce roughness and increase sphericity. The particle size distribution in deposited volcanic ash is correlated with traveled distance: coarser

ashes are found at shorter distances. However, small particles (diameter $<20\text{ }\mu\text{m}$) are often lightly

attached to each other or to the surface of the larger particles through electrostatic forces and capillary

forces generated by fluids within the cloud (such as sulphuric acid H_2SO_4). Such an aggregation con

tributes to the wide variation of grain size distribution commonly found in ash deposits, and the presence

of fine particles near volcanoes (Gilbert et al., 1991). The mineralogy of volcanic ashes determines the mass density and the development of electrical charge

before deposition. Both light ($G_s < 2.8\text{--}3.0$) and heavy ($G_s > 2.8\text{--}3.0$) minerals are present in volcanic

ashes; light minerals dominate and represent between 70% and 95% of the mineral content (Shoji et al.,

1993). Most volcanic ashes contain (Nanzyo, 2004): volcanic glass ($G_s = 2.2\text{--}2.4$), feldspars ($G_s = 2.6\text{--}$

2.7), quartz ($G_s = 2.6\text{--}2.65$), hornblend ($G_s = 3\text{--}3.4$), hypersthene ($G_s = 3.2\text{--}3.9$), augite ($G_s = 3.2\text{--}3.6$),

magnetite ($G_s = 4.5\text{--}5$), biotite ($G_s = 2.9\text{--}3.4$) and apatite ($G_s = 3.1\text{--}3.2$). Volcanic glass predominates

among of primary minerals; it has a poorly ordered structure and exhibits very low resistance to chemical

weathering (Tazaki et al., 1992; Shoji et al., 1993).
Eruption

Cloud of pyroclasts (volcanic

ashes, lapilli, bombs) “Volcanic Ash” $d \sim 0.8-1.7 \mu\text{m}$ < d
particle < cm Composition: silicates and volcanic glass
Other minerals: feldspars, quartz, hornblende, hypersthene,
augite, magnetite Diagenesis Dissolution of parent
minerals Leaching of silica Iron and aluminium content
increases Reprecipitation into new minerals “Volcanic Ash
Soil” $d = 2.0-7.0 \mu\text{m}$ $S = 170-340 \text{ mg}^{-1}$ High water
retention potential New clay minerals: halloysite,
allophane and imogolite Cemented structure wind
transported Time (<20,000 years)

Figure 2. The formation of volcanic ash soils.

Volcanic ashes can be classified by composition, with
emphasis on total silica content. In descending

order of total silica content, volcanic ashes are
catalogued as: rhyolite, dacite, andesite, basaltic
andesite

and basaltic (Shoji et al., 1993). Colombian volcanic ash
soils are derived from andesitic and dacitic ashes

that were rich in plagioclase, volcanic glass, amphibole
and pyroxene and poor in quartz (Arango, 1993).

2.2 Post-depositional processes - “Volcanic ash soils”

Volcanic ash soils are residual deposits that evolve in
situ from volcanic ashes. The diagenetic processes

include the dissolution of base minerals, the precipitation
of new complexes such as clay minerals, and

the development of a new structure, fabric and particle
arrangement. A schematic representation of the

formation of these soils is shown in Figure 2.

Time and climate (precipitation, temperature, humidity and
air flow) determine the evolution of

diagenetic processes. Old and highly weathered deposits are
fine and clayey, while young and lightly

weathered deposits are sandy and coarse. The original mineralogy of volcanic ashes affects the weathering

rate. Acidic rocks such as quartz, feldspars, hornblende and micas are more resistant to weathering than

basic rocks such as olivine, pyroxenes and calcium plagioclase (Townsend, 1985). Water availability

controls dissolution and reprecipitation, the rate of chemical reactions, and the efficient leachage of

soluble components (Townsend, 1985; Chadwick et al., 2003).

As weathering progresses, Si, Ca and Na are leached away, and the Al left behind prevails in the

composition of new minerals (Nanzyo, 2004). In arid areas, soluble weathering products such as Si and

base cations are leached from superficial areas and less soluble components such as Al do not move. In

humid regions, all main elements are leached from the top of the layer; base cations, Si and Al reduce

in amounts that depend on the parent material (Ziegler et al., 2003). In such humid environments, the

formation of gibbsite and aluminosilicate clays is minimized when there are high levels of organic matter

that can preferentially absorb aluminum (Shoji et al., 1993). The recharge of cations on the exchange

complexes increases as rainfall and leaching intensity increase. Leaching determines the rate of loss of

cations and hence the rate of acidification. Even though leaching depends on rainfall, it is more directly

dependent on the effective moisture, which is the relation between water holding into pores and leaching

intensity (Chadwick et al., 2003).

While leaching has controlled the formation of Colombian volcanic ash soil deposits, erosion affects

its accumulation. Therefore, the most abundant and thickest

volcanic ash soil deposits are found in

cold regions while deposits of moderate thickness are found in warm and wet environments (Arango,

1993). Moreover, slightly undulating and planar topographies have contributed to the formation of thick

deposits, while steep areas are characterized by thin deposits due to erosion and stripping. The thickness few meters 10-to-30 m Pale yellow soil, apparently formed by sand and silt size particles. It exhibits plasticity upon remolding. A red stiff and thin seam of iron oxide may be present as a marker of an old water elevation. Dark yellow soil mainly made of high plasticity clay, formed from volcanic ash deposited between 4,000 and 6,000 years ago. The deeper layers were formed from materials ejected between 6,000 to 8,000 years ago. Residual layer originated from weathering of fluviovolcanic materials. It has nodules which are remnants of pyroclasts (diameter > 64 mm). 0.0 m

Figure 3. Typical profile of volcanic ash soils in Colombia. General rule: deeper and older → finer and more clayey.

of volcanic ash soil deposits in the Coffee Zone varies from less than 1 m on 45 ° slopes to 10 to 20 m in

flat areas (Terlien, 1997). Colombian volcanic ash soils exhibit high spatial variability in their physical properties, such as color,

consistency, moisture, mineralogy, and dry density. Coarser grained deposits are found closer to Pereira,

while finer grained deposits are closer to Armenia due to their relative distance to active volcanoes (see

Figure 1). Higher sand content is commonly present in shallower layers (young deposits) and higher clay

contents are found in deeper older layers. High pyroxene content is correlated with sandy textures, while

the abundance of amphibole leads to silty and clayey textures (Arango, 1993). A general soil profile is

shown in Figure 3.

2.3 Stress history

These deposits have not been pre-stressed by overburden. However, seasonal wetting-drying cycles and

associated capillary effects are pronounced in the Coffee Zone. The water table is usually found deeper

than 10 m, within the fluvio-volcanic layer. Seasonal changes in water elevation ranges between 2 m and

4 m. A reddish thin seam of iron oxide is often found as an indicator of a prehistoric water table. A perched

water table above the contact between the layer of volcanic ash soil and the underlying fluvio-volcanic

layer can be found during rainy periods as a result of the marked difference in hydraulic conductivity

between the two layers. The mechanical properties of Colombian volcanic ash soils are determined by their structural forma

tion features rather than by their stress history. In particular, their inherent cementation governs their

mechanical behavior, as discussed in subsequent sections.

3 PHYSICO-CHEMICAL PROPERTIES

The mineralogical composition of volcanic ash soils is characterized by the presence of clay minerals that

are uncommon in sedimentary soils. These minerals include: allophane, imogolite, halloysite, ferrihydrite

or Al/Fe-humus complexes, and opaline silica. Al/Fe-humus complexes and opaline silica form in acidic

environments ($\text{pH} \leq 5$) which are rich in organic matter. Allophane, imogolite, halloysite, and ferrihydrite

form in environments with $5 \leq \text{pH} \leq 7$ and low organic matter content (Ugolini & Dahlgren, 2004). Volcanic glass is one of the main primary components in volcanic ash. It is thermodynamically unstable

and weathers readily. During a typical weathering sequence, volcanic glass transforms into allophane,

halloysite, methahalloysite, kaolinite and montmorillonite (Fisher & Schmincke, 1984; Murray et al.,

Table 1. Mineralogical composition (Location: Manizales. Depth: 7.0 m).

Fraction Test Mineral z= 1.5 m z= 5.5 m z= 7.0 m

Clay fraction X-ray Diffraction Halloysite - $10\text{\AA}$ >50% ~74% ~65%

(diameter <2 μm) Mica - ~9% ~12% Cristobalite 15-30% Traces ~20% Feldspars Traces - - Minerals 2:1, 2:2 - ~14% Traces Non quantifiable Present but non - - material by XRD quantifiable

Coarse fraction Analysis in polarizing Volcanic glass 18% 20% 22%

(50-250 μm) microscope for Feldspar plagioclase 51% 45% 17% petrology Hornblend 21% 24% 8% Hyperstone 4% 4% - Magnetite Traces 4% 49% Biotite Traces 2% 3% Lamprobolite Traces Traces Traces Tuff fragments 6% - - Lithic fragments Traces - -

Note: See similar results in Arango, 1993.

Figure 4. X-ray diffraction patterns for different specimen preparation methods (Location: Manizales. Depth: 7.0 m).

1977). This sequence suggests the predominance of allophane in young volcanic ash soils and of halloy

site in older deposits - for the same parent material and weathering conditions.

The mineralogical analysis of Colombian volcanic ash soils shows the predominance of volcanic

glass, feldspar plagioclase and quartz in the coarser silt and sand fractions (Table 1). The clay fraction

contains hydrated halloysite (>50%), moderate amounts of cristobalite, some traces of feldspars and

material non quantifiable by XRD (Figure 4). It is anticipated that allophane is present in the percentage

Table 2. Volcanic ash soils in Colombia and around the

world - Index properties.

Properties Colombia Other countries

pH 6 India [19] 4.5-6.2 [2] 4.6-5.2 USA [5] 6.1-6.3 [3]
3.6-8 Japan [18] 5.1-5.8 [4] 4.8-7.2 New Zealand [14]
5.7-6.2 Java [22] 5.8-6.7 Hawaii [23]

G s 2.47-2.65 [4] 2.58-2.59 Ecuador [13] 1.92-2.67 [1]
2.67-2.74 Japan [11] 2.50-2.67 [28] 2.28-2.65 Fiji [7]

S a [m² g⁻¹] 170-340 [28] 50-250 Japan [18] 581
Allophane (N₂ adsorption) - [17]

e 2.0- 7.0 [3] 2.4-5.3, 1.5-8 Indonesia [25, 27] 1.1-1.9
[21] 1.9-4.1 allophane predominant India [20] 2.0-2.7 [4]
3.0-5.7, 1.0-6.1 Japan [6, 9] 0.88-3.62 [1] 1.8-6.6 Java
[22] 1.3-3.8 [28]

W 0 80-200% [3] 50-300% Indonesia [27] 16-90% [1] 50-100%
India [19] 29-119% [28] 102-205%, 40-50%, 27-184% Japan [6,
11, 9]

w L 52-64% [4] 70-110% halloysite predominant Indonesia
[25] 60-70% [12] 85-190% allophane predominant Indonesia
[25] 120-250 % [15] 95-107% halloysite predominant New
Guinea [10] 37 -117% [28] 156-165% allophane predominant
New Guinea [10] 179-187% Indonesia [26] 80-213% India [19]
72-159%, 31-40% Japan [18, 11] 105-107% Ecuador [8]

w P 27-33% [4] 55-75%halloysite predominant Indonesia [25]
45-50% [12] 65-150% allophane predominant Indonesia [25]
70-150% [15] 65-73% halloysite predominant New Guinea [10]
25-90% [28] 119-129% allophane predominant New Guinea [10]
139-149% Indonesia [26] 40-100%, 17-20% Japan [18, 11] ~60%
Ecuador [8]

S r 65-88% [28] 50-80% India [16] >95% Japan [6]

γ dry [kN/m³] 8.70 (sandy silt) [21] 4.0-7.2, 3.8-12.7
Japan [6, 9] 10.7 (silty sand) [21] 7.9-9.8 Fiji [7]
7.0-8.4 [4] 4.3-7.6 Java [22] 4.5-13.8 [1] 5.7-13.8 [28]

γ sat [kN/m³] 15.1 (sandy silt) [21] 13.3-14.9 India [19]
16.1 (silty sand) [21] 11-14 Ecuador [13] 12.8-13.1 [28]
21.8-25.7 Japan [9]

Fraction

Sand 5-40% [28] 0-30% * 25-59% ** * New Zealand [20]

Silt 55-70% 52-76% * 23-51% ** ** West Indies [24]

Clay 5-25% 21-42% * 17-19% ** and Japan

Maynard et al., 1997, [6] Kitazono et al., 1987, [7]
Knight, 1986, [8] Mendoza, 1985, [9] Miura, 2003,

[10] Moore and Styles, 1988, [11] Moroto, 1991, [12]
Olarte, 1984, [13] O'Rourke and Crespo, 1988, [14]

Parfitt and Kimble, 1989, [15] Rivera, 2003, [16] Rouse et
al., 1986, [17] Shoji, 1993, [18] So, 1998, [19]

Rao, 1995, [20] Rao, 1996, [21] Terlien, 1997, [22] Van
Ranst et al., 2002, [23] Wada, 1990, [24] Warkentin

and Maeda, 1974, [25] Wesley, 1977, [26] Wesley, 2001, [27]
Wesley, 2003, [28] This study.

of clay material non quantifiable by XRD since the sodium
fluoride test used to identify the presence of

allophane returned a positive result (procedure as in
Fieldes & Perrot, 1966): the indicator paper treated

with phenolphthalein turned red when a small amount of soil
was added indicating that pH exceeds 9

as the sodium fluoride caused the release of hydroxyl ions
from the allophane in amounts much larger

than from other clay minerals.

Grain size distribution analyses (sieve and hydrometer)
show the predominance of silt and clay size

particles (see Table 2 and Figure 5). However these results
need careful interpretation since particle

size distribution results are strongly affected by specimen
preparation in these soils. Drying produces

irreversible changes in structure and the extent of
grinding determines the size of surviving aggregations.

Additionally, clay minerals such as allophane have a strong
flocculating tendency at pH $\approx$ 6, and sodium

hexametaphosphate is only partially effective in dispersing

volcanic ash soils (Wesley, 1973; Maeda

et al., 1977, Rouse, 1986; Shoji et al., 1993; Rao, 1995).
Data in Figure 5 are gathered using sodium

hexametaphosphate (ASTM D422-63 1998) and confirmed with
tests run with different concentrations

of a commercially available dispersant (Colloid 260 made by
Rhône-Poulenc).

The specific surface and pore size distribution of
Colombian volcanic ash soils are investigated using

N₂ gas adsorption and desorption experiments (Nova 1200
Quantachrome). Specimens are placed into

a cell and outgassed at 100 °C for a period of 4 h. During
measurements, the temperature is main

tained constant in a liquid-N₂ bath. The measured specific
surface ranges between $S_s = 147 \text{ m}^2 \text{ g}^{-1}$ and

$S_s = 177 \text{ m}^2 \text{ g}^{-1}$ (BET method). Additionally, the
methylene blue adsorption method was used to obtain

the surface available under wet conditions. The specific
surface values vary between $S_s = 245 \text{ m}^2 \text{ g}^{-1}$ and

$S_s = 360 \text{ m}^2 \text{ g}^{-1}$. Both wet and dry results evidence the
presence of very fine clay particles, in agreement

with the detected halloysite and allophane components.

The equivalent particle dimension calculated from specific
surface values varies between 6 and 9 nm

for spherical particles ($\text{diameter} = 6/(S_s \text{ pG s})$), 4 nm
and 6 nm for prismatic particles (length $\gg$ width,

width = $4/(S_s \text{ pG s})$) and, 2 and 3 nm for thin platy
particles (thickness = $2/(S_s \text{ pG s})$). 0.1 1 10 100 1000
Particle diameter [μm] z=1.5 m z=5.5 m z=7.0 m 0 20 40 60
80 100

C u

m

u l

a t
 i v e
 p
 e r
 c e
 n t
 f i
 n e
 r (b
 y w
 e i g

h t) Clay Silt Sand z=1.5 m z=5.5 m z=7.0 m

Figure 5. Particle size distribution (Location: Manizales - Different depths). Pore diameter [Å] 0 50 100 150 200 250 300 1 10 100 1000 10000 P o r e V o l u m e [c m 3 g 1 Å 1]

Figure 6. Pore size distribution of Colombian volcanic ash soils (Location: Manizales. Depth: 7.0 m). The pore size distribution is evaluated by applying BJH method to the N₂ gas desorption isotherm

data. Results show a peak for pore diameters ~25 nm and ~170 nm (Figure 6). Scanning electron microscopy SEM provides further insight into structural features and the distribution

of particles and pores in undisturbed specimens. SEM images in Figure 7 show clusters of particles

with an equivalent diameter ranging between 2.5 µm and 75 µm. Individual particles become visible at

submicron resolution and consist of randomly oriented halloysite cylinders derived from volcanic glass

weathering; they connect their extremes in a finger-like

pattern. The length of individual cylinders varies

between 0.46 and 0.66 μm and their mean diameter is 0.1 μm . The large pores observed in SEM images (Figure 7a) have an equivalent diameter ranging from 4.5 μm

to 10 μm . Pores between clusters have an equivalent diameter of ~0.5 μm to ~2 μm . At the particle

level, pores are controlled by the arrangement of individual particles and are smaller than 500 nm. The in situ pH is 6.5, in agreement with allophane, imogolite (pH = 5-7, Wada, 1989) and halloysite

formation (pH = 5.7-7.1, Wada, 1990). The cation exchange capacity CEC is determined by Na exchange

at the in situ pH. The CEC increases as the clay content increases with depth: CEC = 11.9 cmol(+)/kg

for the z= 5.5 m specimen, and CEC = 38.7 cmol(+)/kg for the z= 7.0 m specimen. These results are

similar to those reported for volcanic ash soils in Costa Rica (CEC = 20-50 cmol(+)/kg - Wada, 1989)

and Honduras (CEC = 50.0 cmol(+)/kg - Fassbender, 1987). Halloysite, the most abundant clay mineral

in these soils, is responsible for the measured cation exchange capacity. The water retention properties reflect the interaction between the mineral, the fabric and water. The

filter paper, salt solutions and porous plate techniques are used to determine the water retention curves of

Colombian volcanic ash soil specimens. Results are presented in Figure 8 for specimens gathered at two

depths. The pronounced differences between these results confirm the high variability in soil properties

with depth. The characteristic curve for the shallower, coarser and denser specimen (z= 5.5 m) is in a narrow

range of water content ($w_0 = 10\%-43\%$). The characteristic curve for the deeper, finer and more porous

specimen (z= 7.0 m) covers a wide range of water content (w

$\theta = 46\%-110\%$). Both specimens exhibit

moderate changes in suction when the moisture $w > w_p$. The high water retention capacity in these soils

is associated to the high specific surface, pore structure and pore size distribution, and the presence of

Figure 7. SEM images obtained at different magnification (Location: Manizales. Depth: 7.0 m).

allophane minerals that may retain water inside the hollow spherules (Lu & Likos, 2004; van Ranst et al., 2002).

4 STATE AND INDEX PROPERTIES

Volcanic ash soils are characterized by very high void ratio, water retention and high plasticity. Reported

index properties of volcanic ash soils around the world and in Colombia are summarized in Table 2,

including values gathered as part of this study.

Drying and remolding alter the properties and behavior of volcanic ash soils and render the traditional

soil classification methods inadequate (Knight, 1986). In the undisturbed state, volcanic ash soils appear

as a low water content silty sand with little or no plasticity. However, they lose particle bonding when

remolded, their moisture becomes apparent and they exhibit a highly plastic behavior (see also NZ

Geotechnical Society, 2003).

Drying alters plasticity and water retention. The high plasticity in the natural condition decreases

after either air or oven drying as is shown in Table 3. The loss in plasticity depends on the initial

water content, drying temperature, and the duration of desiccation (Warkentin & Maeda, 1974; Maeda

et al., 1977; Wada & Wada, 1977; Lohnes & Demirel, 1983;

Wells & Northey, 1984; Townsend, 1985; σ 40 80 120 160 w o
 [%] 10 100 1000 10000 100000 1000000 S u c t i o n [k P a
] FP - Dekka et al. Salt solutions w p w L w sat z=7.0 m σ
 20 6040 80 1 10 100 1000 10000 100000 M a t r i x s u c t i
 o n [k P a] FP - Dekka et al. FP - Fawcett & Collis-George
 FP - Delage - CERMES Porous plate w p w L w sat z=5.5 m

Figure 8. Characteristic water retention curves for
 Colombian volcanic ash soils (Location: Manizales. Depth:

z= 5.5 m and z= 7 m). Results using filter paper technique
 FP are interpreted following the different listed authors.

Table 3. Drying-induced changes in Atterberg limits.

Properties Condition Colombia Ecuador [2] New Guinea [3]

w L Natural 117% [5] 105-107% Halloysite predominant:
 95-107% 81-94% [4] Allophane predominant: 156-165% 112-183%
 [1] 104-200% [1] 97-111% [1] Oven dried 59% [5] 59%
 Halloysite predominant: 61-65% 74-76% [4] Allophane
 predominant: 44% 57-94% [1] 51-66% [1] 64-70% [1]

w P Natural 90% [5] ~60% Halloysite predominant: 65-73%
 45-50% [4] Allophane predominant: 119-129% 82-113% [1]
 57-98% [1] 73-76% [1] [1] Oven dried 46% [5] 46% Halloysite
 predominant: 46-47% 42-45% [4] Allophane predominant: 42%
 46-80% [1] 49-64% [1] 56-58% [1]

study.

Moore & Styles, 1988; Wells & Theng, 1988; Wesley, 1996;
 Pandian et al., 1993; Shoji et al., 1993;

Wan et al., 2002; Wesley, 2003). The extent of irreversible
 plasticity loss remains unclear.

Plasticity data for Colombian volcanic ash soils are
 plotted in Figure 9. The data fall below the A-line.

While halloysite renders low liquid limit and plasticity,
 the presence of allophane increases the Atterberg

limits of these soils (Rao, 1996).

5 ENGINEERING PROPERTIES

5.1 Zero lateral strain K_0 loading (constrained modulus)

The constrained modulus of Colombian volcanic ash soils is

measured in oedometer tests carried on

undisturbed and remolded specimens made of material
extracted as block specimens (Coffee Zone -

Manizales). Undisturbed specimens are carefully
hand-trimmed to minimize disturbance. Remolded

specimens are formed with oven-dry, crushed material. Tests
were repeated with different size fractions

to explore the effects of the degree of remolding.
Undisturbed specimens are tested at their natural water

content and under saturation condition.

Undisturbed specimens are saturated before loading by
submerging them in water while monitoring

changes in specimen height; no volumetric changes are
registered as capillary forces vanish. Remolded

specimens are saturated at $\sigma'_{v} = 72 \text{ kPa}$ which corresponds
to the in situ vertical stress.

Characteristic results are presented in Figure 10. The $e-\sigma'_{v}$
trend for the undisturbed specimens starts

at a high void ratio ($e = 3.7$). The yield stress is reached
at $\sigma'_{v} = 200\text{--}300 \text{ kPa}$; following from previous

discussions, it is anticipated that this yield stress is
not due to pre-loading but to the physical-chemical

formation history (see also Wesley, 1973 and 2001).

The low stress $\sigma'_{v} < \sigma'_{y}$ compressibility is low ($C_c =$
 $0.19\text{--}0.29$) as inherent cementation inhibits com

pression (there is not suction in saturated specimens). The
application of a high vertical stress $\sigma'_{v} > \sigma'_{y}$

damages interparticle bonds, and the sediments gain the
high compressibility ($C_c = 1.11\text{--}1.22$) expected

for high void ratio clayey soils. Unloading and reloading
follow similar paths and the recompression

coefficient varies between $C_r = 0.08$ and $C_r = 0.12$.

Remolded soil specimens are prepared to attain the highest possible void ratio in order to resemble

the open structure existing in undisturbed specimens. However, it is not possible to repack soil grains at

Figure 9. Plasticity of Colombian volcanic ash soils (Data for Pereira gathered by Uniandes, 2001). 0.5 1.0 1.5 2.0 2.5 3.0 3.5 4.0 10 100 1000 Vertical effective stress σ'_v [kPa] Void ratio Saturation Remolded Undisturbed Saturated Volcanic Ash ($e = 0.8-1.7$)

Figure 10. Void ratio vs. effective stress - Undisturbed specimen saturated at zero confinement and remolded

specimen prepared with ground material passing sieve No. 200 and saturated at $\sigma'_v = 72$ kPa. 10 100 1000 Vertical effective stress σ'_v [kPa]

0.001 0.01 0.1 1 10

C_v

[cm

/ s] $w_o = 119\%$ saturated #200 < #40 #40 < #8 pass #200 Remolded Undisturbed

Figure 11. Coefficient of consolidation vs. confining stress - Data gathered during K_0 loading. (Location:

Manizales. Depth: 7.0 m).

a void ratio higher than $e = 2.2-2.7$ (Figure 10). The $e-\sigma'_v$ behaviour is stress controlled in all the stress

range and remolded specimens exhibit high compressibility ($C_c = 0.35-0.61$).

The role of confining stress on the rate of consolidation is highlighted in Figure 11, where the value of

C_v for each loading stage is plotted versus σ'_v (undisturbed specimens). At low stress $\sigma'_v < \sigma'_y$, consolidation

tion occurs fast and the C_v coefficient varies slightly with stress; its value $C_v = 2.0-2.4$ $cm^2 s^{-1}$ is similar

to silts. Beyond the yield stress $\sigma'_v > \sigma'_y$, the

cemented soil skeleton deteriorates and the consolidation rate becomes markedly slower approaching values similar to clays ($C_v = 0.022$ to $0.056 \text{ cm}^2 \text{ s}^{-1}$).

5.2 Small-strain stiffness - Shear wave velocity

The oedometer tests reported above are performed in a cell instrumented with bender elements to study

the evolution of stiffness in both undisturbed and remolded specimens during loading, unloading and

reloading. Characteristic results are summarized in Figure 12.

Saturated undisturbed specimens show very low V_s sensitivity to effective stress at low stress, $\sigma'_v < \sigma'_y$.

Assuming a power equation $V_s = \alpha(\sigma'_v / \text{kPa})^\beta$, the β -exponent is very low ($\beta = 0.059$), in agreement

with particle cementation (in the absence of suction). When the vertical stress exceeds the yield stress

$\sigma'_v > \sigma'_y$, the β -exponent increases to $\beta = 0.28$. This denotes the sudden structural decementation and

decisive volume change with increased stresses. For comparison, the remolded specimens exhibit high

β -exponent ($\beta = 0.50$) within the applied range of vertical stress.

In spite of the cementation in undisturbed volcanic ash soil specimens (high void ratio), their shear

wave velocity is lower than the shear wave velocity of remolded specimens (lower void ratio) in the

typical stress relevant for engineering applications ($\sigma'_v > 100 \text{ kPa}$). It is important to note that cemented

sands and silts have higher shear wave velocity than their uncemented counterparts at all stress levels.

This unique characteristic of volcanic ash soils remarks their exceptional formation history.

5.3 Shear strength

The shear strength of Colombian volcanic ash soils depends on the degree of diagenesis. The drained

shear strength of saturated undisturbed specimens is studied using direct shear and undrained triaxial

compression tests. These silty specimens (location: Manizales; depth: 5.5 m) have a void ratio $e = 1.34$, $10 \text{ } 100 \text{ } 1000$ Vertical effective stress σ_v [kPa] S_w a v e v e l o c i t y V s [m / s] $\alpha = 100$ $\beta = 0.03$ $\alpha = 28$ $\beta = 0.28$ $\alpha = 125$ $\beta = 0.04$ (a) Undisturbed saturated $10 \text{ } 100 \text{ } 1000$ $10 \text{ } 100 \text{ } 1000$ Vertical effective stress σ_v ' [kPa] S_w a v e v e l o c i t y V s [m / s] Saturation (b) Remolded $\alpha = 176$ $\beta = 0.18$ $\alpha = 20$ $\beta = 0.50$

Figure 12. Shear wave velocity vs. effective stress. (a) Undisturbed specimen saturated at zero confinement.

(b) Remolded specimen prepared with ground material passing sieve No. 200 and saturated at $\sigma'_v = 72$ kPa (specimen

from: Manizales. Depth: 7.0 m).

and a high amount of primary minerals such as volcanic glass and plagioclase feldspar which denote a

young soil. Direct shear test results are shown in Figure 13. The peak shear strength shows a friction angle

$\phi = 32^\circ - 34^\circ$. For comparison, published friction angle and undrained strength data are summarized in

Table 4. Consolidated undrained triaxial test data gathered for saturated undisturbed specimens are shown

in Figure 14 (location: Manizales; depth: 5.5 m). The first yield is observed at $\epsilon_1 \approx 0.2\%$. The brittle

strain-stress response reaches the peak at an axial strain of $\epsilon_1 \approx 2\%$. The critical state line has a slope of

$M = 1.2$ which corresponds to an axial-compression friction angle of $\phi = 30^\circ$. $0 \text{ } 100 \text{ } 200 \text{ } 300 \text{ } 400$ Normal stress [kPa] 0

100

200

300

S h

e a

r s

t r e

s s

[k

P a

] $\phi=32^\circ$ to 34°

Figure 13. Drained direct shear test data - Saturated specimens (Location: Manizales. Depth: 5.5 m).

Table 4. Engineering properties of volcanic ash soils (Colombia and worldwide).

Properties Colombia Other countries

k [m/s] 8.1×10^{-7} - 16×10^{-5} [3] 2.0×10^{-5} Ecuador [4]
 8.1×10^{-7} (sandy silt) [10] 3.0×10^{-5} Costa Rica [4]
 4.4×10^{-6} (silty sand) [10] 1.0×10^{-9} - 1.3×10^{-8}
halloysite Indonesia [11] predominant 7.0×10^{-8} [6] $0.8 \times$
 10^{-9} - 2.7×10^{-8} allophane Indonesia [11] predominant 1.4
 $\times 10^{-6}$ - 5.6×10^{-6} Dominica [8]

$\phi [^\circ]$ 26-43 [5] 30-40 Indonesia [12] 29-39 (silty sand)
[10] 29-41 Dominica [7] 22-29 (sandy silt) [10] 30-38 [6]

q u [kPa] 26-309 [1] 65-200 Indonesia - NZ [12] 25-135 [6]
100-230 Japan [4] 20-160 Japan [5]

V s [m/s] 100-210 [2] 135-175 Japan [9] 100-180 [13]

[5] Moroto, 1991, [6] Olarte, 1084, [7] Rao, 1995, [8] Rao,
1996, [9] Sahaphol and Miura, 2005, [10] Terlien,

1997, [11] Wesley, 1977, [12] Wesley, 2003, [13] This study.

5.4 In situ testing: penetration resistance

In situ SPT data provide further information into the
peculiar properties of Colombian volcanic ash

soils. In general, SPT values are lower than $N = 10$, and they usually range between $N = 4$ and $N = 8$.

The interpretation of SPT results needs careful consideration for volcanic ash soils: the SPT induces a

high strain level that is not representative of common engineering conditions and fails to sense the low σ'_{100} 200 300 400 p [kPa] p [kPa] σ'_{100} 200 300

q [

k P

a]

Figure 14. CU triaxial test data - Saturated specimens (Location: Manizales. Depth: 7.0 m).

compressibility of volcanic ash soils. Maeda et al. (1977) proposed the following correlation between

the N-value of allophanic soils and the bearing capacity $q_a = (2 \text{ to } 2.5)N$. The coefficient is higher than

that found for non-volcanic soils at the same water content (i.e. 1 to 1.3).

5.5 Hydraulic conductivity

A wide range of hydraulic conductivity values has been reported for Colombian volcanic ash soils

(Table 4). In general, volcanic ash soils tend to be more permeable than it is expected for soils with such

a high specific surface (Wesley, 1977). This is related to the intercluster porosity (Figure 7). The hydraulic conductivity values inferred from consolidation data range between $k = 7 \times 10^{-8}$ to

13×10^{-8} m/s for undisturbed specimens at low stress $\sigma'_v < \sigma'_y$. The hydraulic conductivity appears

stress-dependent beyond the yield stress, and it decreases markedly, reaching $k = 1.9 \times 10^{-9}$ m/s at

$\sigma'_v = 636$ kPa. It has been observed that environmental changes modify conduction properties in volcanic ash soils.

In particular allophonic soils develop positive surface charge at low pH and tend to swell; this causes a

decrease in large pores and a reduction in hydraulic conductivity (Ishiguro & Nakajima, 2000). Wet-dry

cycles can have an important effect on hydraulic conductivity of shallow layers through the formation

of fracture networks.

5.6 Electrical conductivity

The electrical conductivity of the pore fluid determines the electrical conductivity of the sediment, yet

reduced by the porosity and the degree of saturation (Archie's law). Ionic concentration and valence

define the pore fluid conductivity. The double layer around minerals adds surface conduction: the higher

the specific surface of the soil the higher the contribution of surface conduction. The electrical conductivity σ E of the pore fluid in Colombian volcanic ash soils varies between

σ E = 0.21 dS/m and σ E = 0.23 dS/m for specimens at z = 5.5 m and z = 7.0 m, respectively (Location:

Manizales). These values indicate low ionic concentration which may be a consequence of good drainage

conditions. The equivalent osmotic pressure derived from electrical conductivity values is π = 7.6 kPa

and π = 8.4 kPa (according to McBride, 1994) and π = 6.0 kPa y π = 6.6 kPa (according to USDA,

1950). S p e c i m e n h e i g h t before saturation (Sr~88%) after saturation after consolidation to σ v = 920 kPa 0 4 8 12 Resistance [k Ω] 4 mm

Figure 15. Electrical resistance profile - Undisturbed specimen under different loading and moisture conditions.

(Location: Manizales. Depth: 7.0 m).

The high resolution electrical profile shown in Figure 15

is measured from the top towards the bottom

of the specimen (height = 4 cm) using an electrical needle probe (diameter: 2.18 mm, Lee et al., 2005).

The uniform profile suggests high homogeneity at the meso-scale.

5.7 Thermal conductivity

The thermal conductivity of dry soils is related to porosity, contact quality, and degree of saturation.

The thermal conductivity k_t of undisturbed Colombian volcanic ash soils is measured using the thermal

needle probe technique (Yun & Santamarina, 2005). Results show an average value of $k_t = 0.71 \text{ W/mK}$

for a water content $w_\theta = 115\%$. The volume fraction of air for this specimen is $V_a/V_{tot} \approx 0.13$, and

the trend by Tang et al. (2005) predicts $k_t \sim 0.85$. Results reported for other volcanic ash soils include:

Japanese allophanic soils $k_t = 0.5\text{--}0.7 \text{ W/mK}$, Japanese andisols $k_t = 0.6\text{--}0.8 \text{ W/mK}$ (Maeda et al., 1977),

and Chilean andisols $k_t = 0.84\text{--}0.94 \text{ W/mK}$ (water content ranging between $w_\theta = 30\%$ and $w_\theta = 33\%$ -

Antilen et al., 2003).

6 ENGINEERING PROBLEMS

6.1 Degradation due to drying and wetting cycles

Dessication effects depend on climatic conditions (evaporation rate, temperature gradients, precipitation,

etc.), water table and vegetation (Naser & Drobroslav, 1995). Colombian soils in the Coffee Zone are

subjected to frequent drying and wetting cycles. Typically, a long rainy period occurs during April

May, and a second one in October-November. Each of these wet periods is followed by a severe dry

period.

These sequential dry-wet periods lead to desiccation crack formation as it is readily observed in all

exposed cuts (Figure 16). The desiccation of a saturated soil and the development of desiccation fractures

progresses in stages:

First, water evaporation increases suction and the soil contracts while the surface remains saturated

(Aitchinson & Holme, 1953). During this stage, the volume change equals the moisture loss, and it is

Figure 16. Gradual degradation of slopes (Pictures taken at Pereira - Armenia road).

determined by the clay content, mineralogy, texture and structure (Aitchinson & Holmes, 1953; Baer &

Anderson, 1997). As the soil gets denser, the resistance to contraction increases and air finally enters into the soil

structure, and desiccation propagates into the soil volume at almost constant volume; this is the upper

limit of residual contraction (Aitchinson & Holmes, 1953; Bronswijk, 1991; Colina & Roux, 2000).

Particle aggregation due to drying transforms microspores into macrospores and results in a reduction

of the water-holding retention capacity. During the last stage of the desiccation, the soil volume remains constant and water loss equals

air volume entering into the soil. The transition between the residual contraction and the null contrac

tion stages is defined as the lower limit of residual contraction (Bronswijk, 1991; Colina & Roux,

2000). The spatial variability of moisture content produces differential suction stresses and local contraction

leading to crack formation. Cracks grow in length as well as in depth until the contraction is insufficient

to cause further crack propagation. New cracks propagate

towards preexistent cracks intersecting them at $\sim 90^\circ$.
Secondary cracks are

usually initiated at the middle of the longest side of a
preexistent block where contraction is highest

(Morris et al., 1992; Jagla, 2004). Therefore, dessication
cracks form a pattern of polygonal blocks (with 4-6 sides)
with a size that is

proportional to the width of the drying layer (Colina &
Roux, 2000; Jagla, 2004). Large cracks are more prone to
form than shorter ones because larger cracks need lower
contraction

to propagate. The contraction across a crack is inversely
proportional to the square of the crack length

$[L]^{-0.5}$. Moreover stresses redistribute in the presence
of adjacent cracks (Morris et al., 1992). Dry conditions
cause irreversible aggregation and crack formation on
slopes in volcanic ash soils. In

fact, altitude and slope orientation towards the sun are
the main factors influencing the deterioration of

slopes in the Colombian Coffee Zone (Imeson & Vis, 1982).
Drying (11 hrs) and wetting (1 hr) cycles are imposed
Colombian volcanic ash soil specimens to

corroborate the effect of dessication cycles on slope
deterioration. Specimens are 50 mm large cubes

trimmed from undisturbed block samples. Results are
summarized in Figure 17. The data in Figure 17-a 0 200 400
600 800 Time (min)

0.6

0.8 1

w

/ w

o 20°C 30°C 40°C 45°C 50°C T (a) 0 2000 4000 6000 8000 Time
(min)

0.4

0.6

0.8 1

w

/ w

o 20°C 30°C 40°C 45°C (Fracture 2 nd cycle) 50°C (Fracture 1 st cycle) No cracks T (b)

Figure 17. Drying and wetting cycles - Damage. (Location: Manizales. Depth: 7.0 m). (a) Moisture lost in once

cycle. (b) Multiple cycles and crack formation.

show the controlling effect of temperature on water loss. The data in Figure 17-b show the effect of

temperature and cycles: no cracks are observed after 9 cycles when drying cycles take place at $T = 20^{\circ} \text{C}$,

30°C or 40°C . However specimens show large fractures after the second drying cycle when $T = 45^{\circ} \text{C}$,

and after just the first drying cycle when $T = 50^{\circ} \text{C}$.
0 10 20 30 40 50 w o [%] 8 10 12 14 γ_{dry} [kN / m³] γ_{max}
=13.3 kN/m³ γ_{dry} (in situ)=10.6 kN/m³ S r = 100% w o (in situ)=34% w opt =25% S r = 90% S r = 80% S r = 70% S r = 60%

Figure 18. Compaction curve obtained with air dried soil, passing sieve No. 40 (Location: Manizales. Depth: 5.5 m).

6.2 Compaction and collapse upon saturation

Figure 18 shows the compaction curve for a relatively young and dense specimen ($e_o = 1.3$ - location:

Manizales, depth: 5.5 m). Older, finer and more porous volcanic ash soils tend to exhibit even more

elusive peaks. In fact, an old volcanic ash soils that have not experienced severe desiccation may not

display a distinct maximum dry density or an identifiable optimal water content. However, even in

this case, the compaction response becomes more conventional after drying and rewetting. Apparently,

suction induced aggregation renders a less plastic material made of silt-size clusters. The natural moisture

content in the field is typically wetter than the optimum water content obtained in the laboratory with

previously dry material. Therefore construction procedures require drying before compaction. Distinct construction techniques have been developed for these soils to control the compaction effort

and the extent of drying. The compaction energy has an important effect: as the number of blows

increases or heavy machinery is used in field, soil strength reduces markedly during compaction as the

soil structure is progressively destroyed and water is released (Wesley, 2003). These observations suggest that conventional compaction specifications are not applicable to these

materials. Dr. Millan (Personal Communication - Pereira) has successfully demonstrated through numer

ous case histories an alternative approach that consists of excavating the borrow material and placing it

with minimal disturbance. Low energy, light compaction equipment is used to attain a density similar to

the original one in situ and at the same natural water content. This methodology is experimentally evaluated using oedometric tests. Figure 19 shows the $e-\sigma'_{v_0}$

response of the specimen compacted to the in situ density and water content. The structural stability

of an identical specimen is evaluated by enforcing saturation at $\sigma'_{v_0} = 100$ kPa. The load-deformation

response is very similar for both specimens and there is only a small volume reduction upon saturation

at $\sigma'_{v_0} = 100$ kPa (collapse index $I_c = \frac{e_0 - e_c}{1 + e_0} = 0.33\%$). Additional tests were run to determine the collapse potential of intact soil (ASTM: D5333, 1996).

Several undisturbed, unsaturated specimens are loaded to

preselected values of vertical effective stress

($\sigma'_{v} = 100$ to 1600 kPa). Then, specimens are saturated. Results in Figure 20 show contraction in all

cases, with a small collapse index $I_c < 1\%$. This low collapse potential confirms the presence of insoluble

cement bonding. $10\ 100\ 1000\ 10000$ Vertical effective stress σ'_{v} [kPa] $0.8\ 1\ 1.2\ 1.4\ 1.6$ V o i d r a t i o Saturation $e_o = 1.3 - 1.4$ Unsaturated, $w_o = 34\%$ $\gamma = 14.2$ kN/m³ Saturated at $\sigma'_{v} = 100$ kPa $I_c = 0.33\%$

Figure 19. Oedometric test for remolded specimens prepared with grains passing sieve #40. One specimen is

loaded at the in situ water content $w_o = 31\%$. The other is loaded to $\sigma'_{v} = 100$ kPa and saturated before further

loading. (Location: Manizales, $z: 5.5$ m). $10\ 100\ 1000\ 10000$ Vertical effective stress σ'_{v} [kPa] $0.8\ 1\ 1.2\ 1.4$ V o i d r a t i o $10\ 100\ 1000\ 10000$ Vertical effective stress σ'_{v} [kPa] $0.01\ 0.1\ 1\ 10\ 100$ C o l l a p s e I n d e x [%] $d < 200$ encircled data correspond to saturation stage Undisturbed ($z = 5.5$ m) $\#40 < d < \#80$ $\#200 < d < \#40$ Remolded specimens

Figure 20. Collapse potential - Undisturbed specimens (Location: Manizales). For comparison values of collapse

index I_c are shown for remolded specimens formed with different size fractions.

6.3 Slope stability

Cementation and suction are important contributors to slope stability in volcanic ash soils. Natural slopes

in these deposits tend to be much steeper than those in sedimentary soils. In fact, natural and man-made

slopes in Colombian volcanic ash soils may exceed 20 m with slopes angles higher than 60° . (Forero et al.,

1999; Redondo, 2003) even though, friction angles are lower than $\phi \sim 34^\circ$. However these slopes and

cuts are susceptible to gradual instability and surface degradation depending on the climatic conditions

and vegetation cover. The orientation of the slope with respect to the sun has a decisive effect. Frequent landslides in the Coffee Zone have produced many deaths and economic losses. The most

common trigger mechanisms are: heavy rainfalls and earthquakes, aggravated by loss of forest cover,

non-planned land use, and poor waste water management. The annual precipitation in the Coffee Zone

ranges between 1500 mm and 2250 mm. Rainfalls higher than 70 mm in one or two days (Terlien, 1997;

Cuadros & Sisa, 2003) triggers shallow landslides with depth <1.5 m. Deeper landslides are triggered

by precipitation higher than 200 mm during 25 days followed by a low daily rainfall (<50 mm - Terlien,

1997). Slope failures also occur after a heavy rainfall during a very dry period (January-February and

July-August). Shallow slope failures in Colombian volcanic ash soils develop a planar, translational slide. Deeper

failures (3 to 10 m) present more irregular slip surfaces. The sliding surface is commonly formed along

the contact between the volcanic ash layer and the underlying fluvio-volcanic deposit (Forero et al., 1999;

Cuadros & Sisa, 2003). Dramatic permeability differences between these layers lead to an increase in

pore pressure in perched water tables (Ng et al., 2003). In addition, the network of dessication fissures

fills during rainfalls and prompts failures. High magnitude earthquakes have high recurrence in the Coffee Zone. Manizales is 15 km away from

seismotectonic zone capable of generating earthquakes at depths ranging between 10 km and 13 km and

Richter magnitudes greater than 6 (Valencia, 1988). In this region, earthquakes have triggered slides in

steep slopes in both residual and colluvial materials (Terlien, 1997). The distance from the epicenter,

frequency and event duration determine the earthquake effect on slope stability. Rodriguez (2002) shows

that slope failures in volcanic ash soil deposits are triggered when the seismic peak velocity of 0.09 m/s

for earthquakes originate in the Subduction Zone, or when the seismic peak velocity exceeds 0.23 m/s

for earthquakes that originate in the Earth's crust (Redondo, 2003).

7 CONCLUSIONS

The evaluation of volcanic ash soils for engineering applications demands a rigorous understanding of

their structure, mineralogy, and formation history. Volcanic ash soils result from the in-situ diagenesis of volcanic ashes. The main processes involved

are the dissolution of original minerals, selective leachage, and re-precipitation of new minerals. Diagenesis produces changes in particle size (from silt and sand size to clay-size), increase in void

ratio (from $e < 1$ in volcanic ash, to $e > 2$ in volcanic ash soils), and changes in mineralogy (from initial

volcanic ash minerals, to allophane, imogolite and halloysite). Volcanic ash soils have a cemented, highly porous granular structure, with low stress dependent

stiffness and low compressibility below the yield stress. Inter-particle aggregation imparts a misleading

silt-like soil appearance. The unique formation history affects all forms of conduction, including hydraulic, thermal and

electrical. Stress-strain behavior varies considerably when soils are disturbed (i.e. overloading, drying, etc.).

Geotechnical problems and construction techniques associated with these materials are directly linked

to the magnitude of the disturbance produced. Traditional classification methods are inadequate for volcanic ash soils. Drying and remolding

practices modify plasticity, stiffness, strength. Likewise, data from in situ tests require careful

interpretation. The water retention potential of volcanic ash soils reflects the formation history. Mature volcanic ash

soils have inter-aggregate and intra-aggregates porosity and high clay content, and develop high suction. High and steep natural and man-made slopes in the field manifest the contribution of both cementation

and suction to soil strength. Drying and wetting cycles cause desiccation cracking and soil deterioration. The effects depend on

temperature and the duration of the drying period. Desiccation cracking is the prevailing mechanism in

the degradation of slopes and cuts.

Compaction causes desegregation, the release of water in pores, and the manifestation of the soil

as a high specific surface clayey sediment. Alternative compaction techniques have been suggested for

volcanic ash soils, including minimal compaction to preserve the in situ soil structure.

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Characterisation and Engineering Properties of Natural Soils - Tan, Phoon, Hight & Leroueil (eds) © 2007 Taylor & Francis Group, London, ISBN 978-0-415-42691-6

Pyroclastic flow deposits from the Colli Albani: The example of

the Pozzolana Nera

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ABSTRACT: This paper presents a selection of the results of an extensive programme of laboratory

testing on intact and reconstituted samples of a pyroclastic weak rock from the volcanic complex of

the Colli Albani (Central Italy). In situ, these deposits are often found in unsaturated conditions; this

has an important practical relevance on the stability of slopes and cuts. The deposit selected for the

investigation is known as Pozzolana Nera and may be assimilated to a bonded coarse grained material.

The nature of bonds and the micro-structural features were examined by means of diffractometry, optical

and electron microscopy. As bonds are made of the same constituents of grains and aggregates of grains,

bond deterioration and particles breakage upon loading are indistinguishable features of the mechan

ical behaviour. The testing programme consisted mainly of one-dimensional and drained and undrained

triaxial compression tests in a wide range of confining pressures. Sustained loading oedometer tests

revealed significant creep. As the confining stress increases, the mechanical behaviour of the material

in triaxial compression changes from brittle and dilatant to ductile and contractant; for both brittle and

ductile behaviour failure is associated with the formation of shear surfaces separating the sample in

several parts at the end of test. The experimental stress-dilatancy relationships were first compared with

the classical stress-dilatancy theories for a purely frictional material and for a material with friction

and cohesion between particles. The analysis of the data indicates that peak strength in the stress-strain

curve does not correspond to the maximum rate of dilation. The testing programme made it possible to

examine the hydraulic and mechanical properties of unsaturated Pozzolana Nera.

1 INTRODUCTION

The behaviour of hard soils and weak rocks started to receive increasing attention during the mid to late

1980s. They have since been the subject of two international conferences (Anagnostopoulos et al., 1993,

Evangelista & Picarelli, 1998) and the subject of much worldwide research interest.

“Structured” hard soils and weak rocks are natural materials resulting from different geological processes, such as cementation, ageing, and overconsolidation, which increase the stiffness and the strength of the corresponding “de-structured” material (Leroueil & Vaughan, 1990, Kavvadas, 1998) or from the progressive physical and/or chemical degradation of a parent rock weakened by weathering or temperature effects. For coarse-grained soils and weak rocks, structure is associated mainly to inter-particle bonding, which may be residual or connected to the precipitation of cementing agents.

A common feature of all these materials is that their behaviour is stiff and brittle at low confining pressures (rock-like) while it becomes softer and more ductile at higher confining stress (soil-like). The extent of the region of the stress space where the behaviour is rock-like is always stress path dependent; however, for bonded granular soils, the size of the rock-like region is mainly controlled by the bond strength rather than by the initial voids ratio, stress state and history, as for sedimentary clays.

The mechanical behaviour of structured hard soils and weak rocks has been extensively investigated in the laboratory over the last decade (Aversa, 1991, Cuccovillo & Coop, 1993, Lagioia & Nova, 1995, Amorosi & Rampello, 1998, Kavvadas & Anagnostopoulos, 1998, Aversa & Evangelista, 1998). In isotropic compression the response of structured soils is very stiff up to a well defined yield state at which large volume strains start to occur (Airey, 1993). Yield results from the degradation of inter-particle bonds and corresponds to the onset of particle

crushing, as suggested in Coop (1999). After

yield, the state of the material approaches a unique normal compression line which results from gradual

deterioration of bonds and increasing particle crushing (Airey, 1993, Coop & Atkinson, 1993). In some

cases, for relatively porous weak rocks, the breakage of bonds implies a sudden collapse of the soil

skeleton at nearly constant effective stress (Lagioia & Nova, 1995). The mechanical behaviour in triaxial compression depends strongly on the confining stress. At low

confining pressures the mechanical behaviour is brittle and dilatant; with increasing confining stress the

behaviour becomes more ductile and contractant, although experimental results indicate that for many

natural weak rocks there is a range of confining stress where a brittle stress-strain curve with a well

defined peak of deviator stress is associated with contractant behaviour (Lagioia & Nova, 1995, Aversa

& Evangelista, 1998). Generally, when the behaviour is dilatant, results from natural and artificially

cemented soils show that the peak stress ratio does not correspond to the maximum rate of dilation

(Elliott & Brown, 1985, Maccarini, 1987). The hypothesis that inter-particle bonding contributes to

the peak strength of structured soils in addition to inter-particle friction and dilation is supported by

classic experimental results showing that the peak strength of artificially cemented sands increases with

increasing percentage of cement (Clough et al., 1981). Upon loading, bond degradation and particle

breakage gradually occur; eventually the mechanical behaviour of the material will be controlled only

by friction between particles and dilatancy due to fabric

and particle interlocking. This paper presents the results of an extended experimental investigation of the structural features and

mechanical behaviour of a pyroclastic weak rock from the volcanic complex of the Colli Albani, typical of

the area south-east of Roma. The engineering interest in the behaviour of the Roman pozzolanas derives

by the need to assess the stability of sub-vertical cuts and underground cavities opened for exploitation

(Cecconi & Viggiani, 2000). The deposit chosen for the experimental study is known locally as Pozzolana

Nera and can be assimilated to a bonded coarse grained material; based on its characteristic value of

unconfined compressive strength (~ 2 MPa) the material is at the transition between soils and rocks. Despite its significant occurrence in the subsoil of Rome, a systematic study of the physical and

mechanical properties of the Pozzolana Nera had not been undertaken until relatively recently; the

geological origin and the main physical and micro-structural properties of the material are described

in detail in the paper. The experimental results highlight the influence of inter-particle bonds and their

progressive degradation on the shear strength of the material; breakage of inter-particle bonds mainly

reduces the cohesion intercept of the failure envelope, while dilation increases the mobilised angle of

friction, to a different extent, depending on the strain level and the effective confining pressure. The main outcomes of this research have been published throughout the recent years (Cecconi, 1999,

Cecconi & Viggiani, 2001, Cecconi et al., 2002, 2003). Some of the experimental results presented in this

and previous papers were obtained in collaboration with many people of different research institutions:

- sampling and microstructural features: M. Marinelli, Università di Roma Tor Vergata; M. Sciotti., P. Tommasi & T. Rotonda, Università di Roma La Sapienza;
- triaxial testing: S. Rampello, Università di Roma La Sapienza; M. Coop, while at GERC, City University;
- grain crushing in shear and isotropic and 1D compression tests: E. Cattoni & V. Pane, Università di Perugia;
- long term behaviour: V. Pane & V. Aneris, University of Perugia;
- hydraulic properties and unsaturated behaviour: E. Cattoni & V. Pane, Università di Perugia; C. Jommi, Politecnico di Milano. Constitutive modelling and theoretical studies were also carried out in co-operation with researchers

from different Institutions, whose support is gratefully acknowledged:

- constitutive modelling and thermodynamics: C. Tamagnini, Università di Perugia; A. De Simone, Max Planck Institute for Applied Mathematics, Leipzig, Germany.

The invitation to submit a paper to this international workshop provides a good opportunity to summarise

the work done to date and define the areas which need further research, with the objective of addressing

relevant engineering applications, such as the stability of underground openings and tunnelling.

2 ENGINEERING GEOLOGY

Pyroclastic deposits cover large areas of Central and Southern Italy for a total extent of 8000 ÷ 9000 km².

The cities of Roma and Napoli are both located in the heart of a wide volcanic region mainly formed

by the volcanic complex of Colli Albani at the south-east of Roma and by the Somma Vesuvio and the

Campi Flegrei districts in the area of Napoli, at the south-east and west of the town respectively.

The main volcanic complexes in the area of Roma are the

Monti Sabatini, 30 km North-West, and the

Colli Albani, about 25 km South-East of the city. The most intense explosive activity of the Colli Albani

dates to the Upper-Middle Pleistocene or about 500,000 years ago. The volcanic complex has been

inactive since that time. More precisely, between 700,000 and 338,000 years ago, the central volcano

erupted a large volume of products, mostly pyroclastic flows, hot masses of rock fragments and gases

moving rapidly along the ground surface in response to gravity, deposited at high temperature. These

deposits smoothed out the pre-existent topography and produced strong alterations of the pre-volcanic

morphology, progressively conditioning the drainage network development, the location of the main

river valleys, and modifying the deep hydro-geological regime of the entire Campagna Romana (De

Rita & Funiciello, 1985). The present geomorphological setting of the Colli Albani is shown in Figure 1.

According to De Rita et al. (1988), the volcanic activity of the Colli Albani occurred in three successive

phases which differ basically in their eruption mechanism. The first phase (Tuscolano Artemisio stage)

which started about 700,000 years ago, covers more than half of the entire active life of the volcano and

resulted in the deposition of 280 km³ of products, see Figure 2. This phase ended up with the collapse of

the caldera of the central edifice. The Tuscolano Artemisio phase includes four cycles during which large

pyroclastic flows and subordinate fall-deposits and lava flows were put in place. During the third cycle,

two pyroclastic flow units, named "Tufo Lionato" and "Pozzolane Nere" were deposited; these units

include a loose ashy matrix and a lithoid facies. The deposits of "Pozzolanelle" and "Tufo di Villa Senni"

were then erupted during the fourth cycle of Tuscolano Artemisio. Also these units include two facies,

a loose ashy matrix and a lithoid facies. After a short interruption, the volcanic activity started again

inside the collapsed area with the inception of a new small central edifice (Campi di Annibale phase).

Finally the Colli Albani volcanic complex came to an end with several hydromagmatic explosions from

eccentric craters in the western sector (final Hydromagmatic phase).

The typical subsoil profile from the south-east of Roma is shown schematically in Figure 3(a); the

thickness of each unit can vary from a few tens of millimetres to some tens of meters. Generally,

pyroclastic deposits contain crystals, glass shards and pumices, and lithic fragments in highly variable

proportions, depending on the type of eruption and the chemical composition of the magma. Individual

layers generally lack internal stratification, although the superposition of a number of flow units may

give the appearance of organised bedding.

Figure 1. Virtual computerised reconstruction of the Colli Albani (www.italyexplorer.it).

Figure 2. Simplified geological map of the area of Rome including the Colli Albani (from De Rita et al., 1992).
Key:

(1) travertine; (2) Plio-Pleistocene sedimentary units; (3) final hydromagmatic units; (4) air fall deposits; (5) lava

flows; (6) pyroclastic flow units of the Colli Albani; (7) pyroclastic flow units of the Sabatini volcanic field; (8)

Tortonian flysch; (9) caldera rims; (10) late explosion craters; (11) Meso-Cenozoic pelagic carbonate units; (12)

Meso-Cenozoic carbonate platform units.

Figure 3. (a) geological profile of the Colli Albani volcanic complex (adapted from De Rita et al., 1988);

(b) sub-vertical cut in a quarry of Pozzolana south east of Roma (Fioranello quarry).

Pozzolanas have been continuously exploited since Roman and Etruscan times to produce hydraulic

mortars and cement; they are also often employed as the underlying stratum of road pavements and tennis

courts. The prolonged mining activities have resulted in a complex network of underground cavities; in

some cases sudden collapses have occurred with the formation of sinkholes at the surface (Bernabini

et al., 1966, Lembo Fazio & Ribacchi, 1990). Open quarries of pozzolanas are also frequent in the south

east of Roma, with subvertical excavation fronts which can be as high as 20-25 m. However, there are

underground cavities not yet surveyed and mapped.

The Pozzolana Nera, which was selected for this experimental investigation, was extracted from the

toe of a 23 m high vertical cut shown in the photograph in Figure 3(b), in a quarry located about 10 km

South of Roma. At the site the thickness of the Pozzolana Nera is about 10 m; the degree of fissuring of

the rock mass is generally quite low, without any detectable families of discontinuities. The deposit is

above water table and therefore partially saturated in its natural state, with an average water content w_c

of about 13 ± 1.7 per cent, which corresponds to a saturation degree S_r of about 43.5 ± 3.5 per cent.

3 COMPOSITION, STATE AND STRUCTURE

Structural features, mineralogical composition and texture

of the Pozzolana Nera were examined by

means of optical and electronic microscopy; X-ray diffraction was also used to identify soil minerals at

the crystalline state.

The Pozzolana Nera has a clastic texture, diffused in a black scoriaceous matrix containing frequent

crystals of leucite, piroxen, biotite as well as lithic fragments of lava. Figure 4 shows the matrix of the

material in plane-polarised light. Crystals of leucite, recognisable from their straight edges, can reach

dimensions of up to 1.5 mm side; voids appear as white areas with no defined edges, see Figure 4(a). It

is generally very difficult to discern grains from aggregates of grains; in some cases it is hard to identify

individual particles at all; in thin section the material may look like a foam, probably due to the process

of deposition, see Figure 4(b). Pore space features such as size, shape, and orientation are very variable;

the closed porosity is very small as confirmed by measurements of specific gravity carried out using a

helium picnometer on increasingly finer powdered material (Cecconi, 1999).

Figure 5(a) shows the clastic texture of the Pozzolana Nera as observed by scanning electron

microscopy on titanium-coated dried samples. The microstructure consists of sub-angular grains of very

variable size with a rough and pitted surface. Figure 5(b) shows a typical vesicle of glass in more detail.

An example of the contacts between two particles at magnification factors of 72× and 1800× is shown

in Figures 6(a) and (b) respectively; a physical bridge is observed bonding the grains, which are evenly

coated by microspherules with a minimum diameter of about

0.5 μm .

In order to identify the nature of bonding, chemical micro-analyses of selected areas of the samples

were performed in the electron microscope pointing at particles and bonds between particles. No sig

nificant difference in the mineralogical composition was revealed by the analyses, which indicates that

deposition of cementing agents, such as calcite or silicates, did not occur; furthermore, no secondary

minerals such as zeolites, very common for pyroclastic soft rocks, were detected. The main minerals at

the crystalline state were identified as Leucite, Magnetite, Augite and Halloysite (see Figure 7).

Figure 4. Thin sections of Pozzolana Nera in plane polarized light.

Figure 5. Scanning electron micrographs of Pozzolana Nera (a) 200 $\times$; (b) 3000 $\times$.

Figure 6. Scanning electron micrographs of Pozzolana Nera: (a) 72 $\times$ and (b) detail at 1800 $\times$. L H L L H M A H A Augite H Halloysite L Leucite M Magnetite 2 θ ($^\circ$)
2 12 22 32 42 I

Figure 7. X-ray diffractometer chart of Pozzolana Nera.
Table 1. Average values of the physical properties of Pozzolana Nera. w c γ γ d S r (%) G s (kN/m³) (kN/m³) n (%) 13 $\pm$ 1.7 2.69 16.46 $\pm$ 0.06 14.64 $\pm$ 0.03 0.45 $\pm$ 0.01 43.5 $\pm$ 3.5 0.01 0.1 1 10 100 diameter (mm) 0 10 20 30 40 50 60 70 80 90 100 p e r c e n t a g e p a s s i n g (%)
silt sand gravel

Figure 8. Grain size distribution of Pozzolana Nera.

Intrinsic inter-particle bonds are probably due to the original material continuity partially still visible

in the foam microstructure revealed by thin sections. Hence bonding is connected to the mechanism of

deposition, but is not associated to diagenetic factors, i.e. chemical alteration of the constituent minerals,

leading to lithification by formation of hydrated aluminosilicates (zeolites).

At the scale of the laboratory sample the material is quite heterogeneous. Values of average physical

properties are summarised in Table 1. Values of voids ratio and inter-particle porosity were determined by

conventional methods and mercury porosimetry; they are rather low, if compared with other pyroclastic

soils and weak rocks (Aversa & Evangelista, 1998, Esposito & Guadagno, 1998).

The grain size distribution depends strongly on the techniques adopted to separate larger aggregates

before sieving and on the methods and time employed for sieving (e.g. Lee, 1991). In this case the grain

size distribution was determined by dry sieve analysis; the sieving time did not affect the results. Before

sieving, the material was broken by hand while immersed in water with a detergent additive to allow

separation of small aggregates, and then oven-dried at 105 ° C. The Pozzolana Nera is well graded and

can be classified as a gravel with sand with a coefficient of uniformity $U = 33.6$, $D_{10} = 0.12$ mm, and

$D_{50} = 2.5$ mm. The grain size distribution is shown in Figure 8.

4 ENGINEERING PROPERTIES

4.1 Experimental apparatus and procedures

The experimental work consisted mainly of one-dimensional compression tests and triaxial compression

tests on intact and reconstituted samples. One-dimensional compression tests were carried out in an

oedometer cell with reduced cross section (35.7 mm dia.) which made it possible to achieve a maximum

vertical stress of 12.8 MPa. Natural samples for reduced

section oedometer tests were cored in areas of the blocks that did not include large lithic fragments.

The majority of triaxial tests were performed in conventional triaxial cells modified to allow computer control of axial and radial stress reaching a maximum cell pressure of 3.2 MPa; a smaller number of tests were carried out in hydraulic stress-path computer controlled cells (Bishop & Wesley, 1974). Three special tests were carried out in high pressure triaxial systems reaching confining pressures of 10, 14 and 70 MPa respectively (Taylor & Coop, 1993, Amorosi & Rampello, 1998, Cuccovillo & Coop, 1998). Axial and volume strains were measured using external displacement transducers and a volume gauge;

in a limited number of tests a pair of miniature lvdt's were mounted directly on the centre length of the sample to measure axial strains locally (Cuccovillo & Coop, 1997). A radial strain belt was used in tests carried out on large samples (100 mm dia.) to measure radial strains. Intact samples of Pozzolana Nera for laboratory testing were obtained by rotary coring partially saturated natural blocks (Cecconi, 1998). Before coring, the blocks were immersed in water, which increased the saturation degree up to $S_r \sim 0.7$, and then frozen at -20°C ; the ends were then squared on a band-saw up to a tolerance of 0.2 mm which, considering the grain size distribution of the material, is the smallest achievable. The effects of freezing on the microstructure of the material were partially investigated by means of compression wave velocity measurements as the velocity with which stress waves are transmitted through a soil depends on the elastic properties and density of the material. The velocity of propagation

of P-waves as been traditionally related to the degree of fissuring and micro-cracking of rock masses

(Goodman, 1989). In this case a significant reduction of the velocity of propagation of p-waves would

indicate a reduction of the initial stiffness of the material probably due to the de-structuring induced by

freezing. Measurements of P-wave velocity were first made on vacuum-dried natural samples and then

repeated on the same samples after they had been immersed in water, frozen at -20° ° thawed and vacuum

dried. The procedures and the times for saturation and freezing were the same as those employed for the

preparation of laboratory samples. P-waves were transmitted and received across unconfined samples

using electromechanical 15 mm diameter transducer with a characteristic frequency of vibration of 1 MHz

under an axial load of 0.3 kN. The results showed systematically the effects of freezing on the stiffness

properties of the material to be very minor, with a reduction of the P-wave velocity of about 6%. Prior to testing, the samples were thawed at room temperature and mounted on the base pedestal

of the cell; the samples were saturated in the triaxial cell under a back pressure of 0.4-0.5 MPa at a

constant mean effective stress $p'_{0} = 0.1-0.2$ MPa; the in situ vertical effective stress was estimated to

be $\sigma'_{v} \sim 0.3$ MPa. The values of Skempton's pore pressure parameter, B, after about a week were still

relatively low (0.50-0.95); however, taking into account the stiffness of the material and allowing for

membrane penetration (Baldi & Nova, 1984), these values correspond to degrees of saturation close to

unity ($S_r = 0.93-0.99$). Drained and undrained tests were carried out following isotropic compression at increasing values

of mean effective stress in the range 0.05-58 MPa. At the end of the compression stage the samples

were kept at constant effective stress for 24 hours before shearing; in all cases the axial strain rate

before shearing was less than 0.005%/h. Drained and undrained shearing were carried out strain con

trolled at displacement rates of about 0.00244 mm/min ($\sim$ = 0.2%/h) and about 0.0122 mm/min ($\sim$ = 1%/h),

respectively. Reconstituted samples were prepared using the granular material obtained with the same procedures

adopted for grading analysis. The material was oven-dried, mixed with de-aired distilled water at a water

content $w_c \sim$ = 20% and compacted in a mould in thin layers. This was done on the fractions of the material

passing sieves astm 5, 10, 18, 35 (4, 2, 1, 0.5 mm) which resulted in increasingly finer and more uniform

samples.

4.2 One-dimensional and isotropic compression

Figure 9(a) shows typical results of one-dimensional compression tests on intact samples of Pozzolana

Nera, as voids ratio, e , versus logarithm of vertical effective stress, σ'_v . The average initial voids ratio of

most intact samples is $e_0 = 0.805 \pm 0.04$. The initial response on first loading is relatively stiff; gradual

yield occurs as the vertical effective stress increases. The values of the yield stress were determined

conventionally using Casagrande's procedure and are shown as arrows in Figure 9(a). For vertical effective

stresses in excess than about 5 MPa, large volume strains occur along a single normal compression line

(1d-ncl), shown as a solid line in Figure 9(a), characterised by a compression index $C_c = 0.305$ and a

voids ratio, e (1 MPa) = 0.935. Unloading shows that irreversible strains occur even at stress levels relatively

far from major yield; the response on unloading is always very stiff with an average swelling index

$C_s = 0.0075$. Yield on reloading occurs gradually as the state of the material approaches again the 1d-ncl. A smaller number of intact samples have an initial voids ratio which is above the range defined by

most samples. Their behaviour in one-dimensional compression is shown in Figure 9(b); in the same

figure, the data presented in Figure 9(a) are also reported for comparison. The response on first loading

of the samples with higher initial voids ratios is less stiff than that of most intact samples, while the state

of the material goes beyond the 1d-ncl defined in Figure 9(a). The average yield stress is approximately

the same as before; an average yield stress for all intact samples is $\sigma'_{vy} = 3.95 \pm 0.67$ MPa. After yield, $0.1 \leq \sigma'_{vy} \leq 1.0$ MPa

0.5

0.6

0.7

0.8

0.9

1.0

e 1D-NCL σ'_{vy} a) $0.1 \leq \sigma'_{vy} \leq 1.0$ MPa 1D-NCL σ'_{vy} b) $0.1 \leq \sigma'_{vy} \leq 1.0$ MPa 1D-NCL PNED7 PNED6 c)

Figure 9. Results from one-dimensional compression tests on (a) intact, (b) intact with higher initial voids ratio,

and (c) intact atypical samples. $0.1 \leq \sigma'_{vy} \leq 1.0$ MPa 0.5 0.6 0.7 0.8 0.9 1.0 1.1 e RIMED1 intact reconstituted 1D-NCL

Figure 10. Results from one-dimensional compression tests on reconstituted samples at different initial voids ratios.

the compressibility of the samples with higher initial voids ratios is larger than that of the majority of

intact samples and the state of the material seems to converge towards the previously defined 1d-ncl.

The chain-dotted straight line in Figure 9(b) is an envelope to the compressibility curves, characterised

by a slope $C_c = 0.425$. This line crosses the 1d-ncl at a vertical effective stress of about 27 MPa.

Figure 9(c) shows two atypical test results; in this case the initial voids ratio is within the range defined

by the majority of intact samples. The response of the material on first loading is very stiff in a wider range

of stresses $\sigma'_v = 0.05\text{--}4$ MPa. As in the previous case, the first loading compressibility curves go beyond

the 1d-ncl defined by the majority of intact samples. At the maximum vertical effective stress attained

in the test the state of the material had not reached a well defined post-yield compression line, therefore it

was not possible to determine the yield stress using the Casagrande's procedure. Although all the results

presented in Figure 9 are relative to intact samples, tests pned6 and pned7 are atypical, in that for a

given value of the vertical effective stress σ'_v , the material exists at higher voids ratios than those possible

for other intact samples with the same initial porosity, which may be evidence of a stronger structure.

The results of tests on five reconstituted samples are given in Figure 10. Four of the samples

were prepared following the procedures described in the previous section (full squares); these sam

ples were partially saturated at the beginning of the test ($S_r = 0.5\text{--}0.6$) and reached saturation during

one-dimensional compression. The open squares refer to the results of one test on a reconstituted sam

ple obtained after consolidation of the finest fraction (0.25 mm) in a consolidometer, under a nominal

maximum vertical stress of 0.1 MPa; due to the reconstitution procedure this sample was saturated from

the beginning of the test. The smallest initial voids ratio that it was possible to achieve for the reconstituted samples by com

paction in the mould, $e_0 = 0.949$, was above the range of values measured for intact samples. The

behaviour of reconstituted samples on first loading is less stiff than that of intact samples. The average

yield stress is approximately the same as the majority of intact samples, $\sigma'_{vy} = 3.96 \pm 0.5$ MPa. After

yield, due to the initially high voids ratio, large volume strains are observed. Due to the initial full satur

ation of the material, the behaviour on loading of the sample prepared in the consolidometer ($e_0 = 0.849$)

is significantly less stiff than any other intact or reconstituted samples. For vertical effective stress in

excess of about 5 MPa, this sample defines a normal compression line which is parallel to the 1d-ncl

defined by intact samples, although at lower voids ratios ($e(1 \text{ MPa}) = 0.857$). Data from isotropic compression tests performed at high ($p'_0 = 8.3$ MPa) and very high ($p'_0 = 58$ MPa)

confining pressures on a reconstituted (RIMAP) and an intact (BPVHP) sample, respectively, are shown

in Figure 11 as specific volume, v , versus logarithm of mean effective stress, p' . The pattern of results

is consistent with that obtained from one-dimensional compression tests. At large stresses the com

pressibility of both samples is described by a unique

straight line characterised, as for one-dimensional compression tests, by a slope $C_c = 0.425$. The set of data presented above shows that the initial porosity is the main factor controlling the compressibility of the Pozzolana Nera. Figure 12(a) shows the values of compression index, C_c obtained from tests on intact and reconstituted samples versus their initial voids ratio; C_c increases approximately linearly with e_0 , consistent with experimental data obtained for other natural soils and weak rocks (Vaughan, 1988). In Figure 12 (b) the data are plotted as vertical yield stress versus initial voids ratio. The data from samples reconstituted by compaction in the mould indicate a reduction of yield stress with increasing initial voids ratio, as observed in isotropic compression on artificially cemented soils (Maccarini, 1987), volcanic weak rocks (Aversa & Evangelista, 1998), and cemented carbonate sands (Airey, 1993, Lagioia & Nova, 1995). On the other hand, the vertical effective stress at yield for intact samples and for sample rimed 1 do not show a clear trend with the initial voids ratio. Intact samples generally have a relatively low initial porosity and are characterised by a stiffer behaviour on first loading than samples reconstituted by compaction in the mould. Gradual yield and normal compression result from progressive breakage of inter-particle bonds and/or grains and aggregates of grains, as bonds and grains are made of the same material. The main effect of the technique used to reconstitute the samples is that of giving an open structure to the material with a higher initial porosity

	0.01	0.1	1	10	100	1000	p' (MPa)	1.3	1.5	1.7	1.9	2.1	2.3	$v = 1 + e$
ISO-NCL														
BPVHP														
RIMAP														

Figure 11. Results from high and very high pressure

triaxial isotropic compression tests.

than the intact samples. Some of the inter-particle bonds are broken by the procedure of remoulding;

residual bonds and fabric, however, allow the state of the reconstituted material to exist beyond the

1d-ncl. A consistent increase in compressibility with initial voids ratio has been observed for intact and

reconstituted samples, although before yield intact samples are always stiffer than reconstituted ones.

4.3 Creep

Like other Italian pyroclastic soils and weak rocks (Croce, 1954, Evangelista & Aversa, 1994) the

Pozzolana Nera may exhibit significant long term deformation due to creep. This was investigated by

performing conventional oedometer tests on both natural and saturated reconstituted samples, these latter

with a grading corresponding to that of a medium sand (Cecconi et al., 2005b). In this section the main

findings about the secondary one dimensional compression of Pozzolana Nera are summarised very

briefly.

Figure 13 shows some typical results - in terms of axial strain vs. time - obtained from a test performed

at increasing values of σ'_{v0} , performed maintaining each loading step for 3 to 5 days. The rate of strain

during primary creep was examined through the secondary compression index on first loading, C_{α} , the

secondary swelling index on unloading, $C_{\alpha s}$, and the secondary re-compression index on re-loading.

(a) 0.6 0.7 0.8 0.9 1.0 1.1 1.2 e 0

0.10

0.20

0.30

0.40

0.50

0.60

C c intact intact, higher e 0 RIMED 1 reconstituted (b) 0.6
0.7 0.8 0.9 1.0 1.1 1.2 e 0 0 2 4 6 8 σ'_{vy} (MPa)
intact intact, higher e 0 RIMED 1 reconstituted

Figure 12. Effects of initial voids ratio on: (a)
compressibility and (b) vertical yield stress. 0 10000
20000 30000 t (min) 0.044 0.042 0.040 0.038 0.036 0.034 e a
18 20 22 24 26 T e m p e r a t u r e (° C) 2nd Jan 5th
Jan $\Delta T_{max} = 6.6$ ° C 7th Jan 20th Dec oed01rec σ'_{vy}
= 100kPa

Figure 13. Axial strain and temperature vs. time.

(a) 100 1000 σ'_{vy} σ'_{vy} C α , C α_s , C α_r ($\times 10^{-3}$)
oed02rec 1 2 3 4 5 6 8 9 10 11 12 13 7 a) 10 6 2 -2 (b) 100
1000 1 2 3 4 5, 9 6 7 8 10 12 11 oed05nat b)

Figure 14. Compression and swelling indexes observed during
loading-unloading cycles for: (a) reconstituted

samples (b) natural samples.

Figure 15. Variation of C α with σ'_{vy} on first loading for
natural and reconstituted samples.

These were conventionally estimated as the slope of the
e-logtcurve in the logarithmic cycle following

the end of primary consolidation (t_{90}) or primary
swelling (Mesri et al., 1978):

Time t_{90} was computed using the method suggested by Taylor
(1948) and varied systematically in the

narrow range 10-30 s. After primary consolidation, during
the log-cycles following the first one after

t_{90} , the secondary compression and swelling indexes
remained approximately constant with time. No

significant effects of temperature were observed, see

Figure 13, where a noticeable temperature variation

of about 7 °C does not induce significant variation of the axial strain rate with time. Figure 14 shows the typical variation of secondary compression, swelling and recompression indexes

with vertical effective stress for (a) reconstituted and (b) natural samples. In both cases, the loops obtained

in the values of the secondary swelling and compression indexes match the loading and unloading cycles

performed during the tests. However, the amplitude of the loops obtained in the tests on the reconstituted

samples is clearly larger than that obtained for the natural samples. This is further evidence of the

existence of inter-particle bonds inhibiting creep for the natural material. Figure 15 shows the variation of C_α with vertical effective stress on first loading, confirming that

the secondary compression index of natural samples is smaller than that obtained for the reconstituted

material. The experimental data are fitted by a power function, Table 2. Creep parameters α and β . Reconstituted samples Natural samples Range Average Range Average α 0.37-0.48 0.42 0.37-0.58 0.50 β $1.3 \cdot 10^{-4}$ $-4.1 \cdot 10^{-4}$ $2.6 \cdot 10^{-4}$ $3.5 \cdot 10^{-5}$ $-15.6 \cdot 10^{-5}$ $7.9 \cdot 10^{-5}$ 0 2 4 6 8 10 q (MPa) ϵ_s (%) -8 -4 0 4 0 4 8 12 16 20 24 8 ϵ_v (%) (a) (b) p'_o (MPa) 2.85 1.40 0.20 2.50 0.70 0.10 2.00 0.35 0.05 (a) (b)

Figure 16. Drained triaxial compression tests on intact samples at low and intermediate confining pressures: (a) $q:\epsilon_s$,

(b) $\epsilon_v:\epsilon_s$.

In which β and α are fitting parameters, whose numerical values depend on the units adopted to

express σ'_v . Their ranges of variation and their average values if σ'_v is expressed in kPa are reported in

Table 2.

Finally, a relationship between the local compression

index, C_{cl} , and the secondary compression

index, C_{α} , on first loading was proposed. Despite of the significant variation of both C_{α} and C_{cl} , the

ratio C_{α}/C_c is reasonably constant at:

$C_{\alpha}/C_c = 0.06 \div 0.07$ (natural) and $C_{\alpha}/C_c \approx 0.04$ (reconstituted)

These values are in good agreement with those suggested in the literature for Italian pyroclastic soils and

weak rocks (Pellegrino, 1967, Penta et al., 1961) and other natural deposits (Mesri et al., 1978, Mesri &

Godlewski, 1977) although higher than those quoted by Mesri et al. (1995) for granular soils.

4.4 Stress-strain behaviour in triaxial compression

Typical stress-strain curves from conventional drained triaxial compression tests on intact samples at

confining pressures in the range $0.05 < p'_{\theta} < 2.85$ MPa are shown in Figure 16(a) in terms of deviator $\sigma_1 - \sigma_3$ versus shear strain, ϵ_s (%). The corresponding curves of volume strain, ϵ_v (%), versus shear strain, ϵ_s (%), are shown in Figure 16(b). The initial specific volume of all triaxial samples was in the range $v_0 = 1.75-1.89$.

Figure 17. Drained triaxial compression tests on intact samples: $\sigma_1 - \sigma_3$ versus ϵ_s .

stress, q , versus shear strain, ϵ_s ; the corresponding curves of volume strain, ϵ_v , versus shear strain, ϵ_s , are

shown in Figure 16(b). The initial specific volume of all triaxial samples was in the range $v_0 = 1.75-1.89$.

The maximum mean effective stress attained in the compression stage was not sufficient to cause major

yield; before yield the behaviour of the material in isotropic compression is very stiff, so that at the start

of shearing the specific volume was close to the initial value. At low confining pressures ($0.05 < p'_{\theta} < 0.70$ MPa) the behaviour is brittle with the deviator stress

increasing roughly linearly up to a sharp, well defined peak which is attained at shear strains in the range

0.4-1.2%; the average initial stiffness, $\delta q / \delta \epsilon_s$, over a range of shear strains $0.1 < \epsilon_s < 0.25\%$ does not

significantly depend on the applied confining pressure. After a small initial compression, the behaviour is

dilatant throughout the test. The maximum rate of dilation decreases with increasing confining pressure;

the peak deviator stress occurs before the maximum rate of dilation, as observed for other soft rocks such

as tuffs (Aversa, 1991, Aversa & Evangelista, 1998), calcarenites (Cuccovillo, 1995), porous limestones

(Elliot & Brown, 1985), and artificially cemented soils (Maccarini, 1987). Peaks are followed by a rapid

decrease of the deviator stress towards an ultimate state where the rate of dilation is zero. At the end of

the test, well defined shear surfaces were observed which separated the sample in two or more nearly

rigid bodies. At larger confining pressures ($1.40 < p' < 2.85$ MPa) the stress-strain behaviour is ductile and the

samples compress throughout the test. The stress-strain curves are initially roughly linear up to a relatively

well defined yield point, with an abrupt change of slope which occurs at shear strains similar to those

corresponding to the peaks at lower pressures. The extent of the nearly-linear portion of the curves

decreases with increasing confining pressure, making it more difficult to define the yield point. After

yield, the deviator stress increases to reach a gentle peak at shear strains of about 7 to 13%; the strain

softening behaviour is accompanied by positive volume strains. On removal of the membrane, the samples

separated in several parts. In Figure 17 the data are plotted in terms of stress ratio, $\eta = q/p'$, versus shear

strain, ϵ_s . From the

tests at low confining pressures ($0.05 < p' < 0.70$ MPa) it is not possible to identify a well defined value

of mobilised stress ratio at the end of the test, $\eta = M$; in some cases, towards the end of the test, the

stress ratio increased as shearing progressed. For these samples, however, the behaviour was associated

with the formation of well defined slip surfaces; therefore, at larger strains, stress-strain data can not

be directly computed from applied forces and displacements measured at the boundary of the sample.

Also, once the slip surface has formed, significant experimental errors in the measurement of the axial

force may be introduced by increased membrane restraint and shaft friction on the axial ram. To a lesser

extent, the same effects may influence the interpretation of the tests carried out at higher confining

pressures ($p' = 1.40$ – 2.85 MPa) for which the strain-softening portion of the stress-strain curves must

be regarded with caution. At these pressures, however, the samples mobilise a relatively well defined

value of the stress ratio $M \approx 1.6$, corresponding to a value $\phi'_{cs} = 39^\circ$. A slight tendency of M to decrease

with increasing confining pressure may be observed.

Figure 18. Thin section of intact sample after shearing at constant $p' = 8.3$ MPa. p' (MPa) 0 0 1 2 3 4 5 6 7 8 2 4 6 8 10 12 q (MPa) peak end-of-test CID CIU

Figure 19. Triaxial compression tests on intact samples at low and intermediate confining pressures: peak and end-of-test stress states.

Figure 17 also shows the curves obtained from two tests carried out at constant mean effective stress

$p' = 8.30$ (bphp) and 58 MPa (bpvhp). The observed

behaviour is consistent with that resulting from

tests performed at relatively large confining pressures. Sample bphp failed in a ductile manner although

at the end of the test an intense state of fracturing was observed. Figure 18 shows a thin section of the

sample after shearing with a fracture following the edges of one of the crystals scattered in the rock

matrix; the presence of crystals in the intact material which are out of the axis of symmetry of the sample

localises strains (Desrues et al., 1996). For test bpvhp the behaviour is non-linear from the very start

of the shearing stage, due to major yield having occurred in the previous isotropic compression stage

(see Figure 11). For this test the stress-strain curve still shows a gentle peak at shear strains of about

16%, which is likely to correspond to the activation of a failure surface, clearly visible on removal of the

membrane at the end of the test.

The stress ratio mobilised at the end of the four undrained triaxial compression tests on isotropically

consolidated samples tends towards the same value, $\eta = M = 1.6$, which was also defined by drained

tests at relatively large confining pressures. For all undrained tests the behaviour of the soil skeleton is

initially contractant with positive excess pore water pressures; the maximum value of u corresponds to

yield at maximum stress ratio. After yield a decrease in pore water pressures is observed corresponding

to a tendency of the soil skeleton to dilate.

4.5 Strength

The shear strength of the Pozzolana Nera was examined in terms of stress parameters q , p' . Figure 19

shows the stress states at peak deviator stress and at the end of the test for intact samples tested at low

and intermediate confining pressures; the effective stress paths of the undrained tests are also shown in v - $\ln p'$ plot for p' (MPa) 1.3 1.5 1.7 1.9 2.1 2.3 v CSL ? CID CIU initial state end-of-test RIMAP $\psi < 0$ $\psi > 0$

Figure 20. Paths followed during shear and end-of-test states for all intact samples. ϕ (degrees) 0.01 0.1 1 10 100 diameter (mm) 0 10 20 30 40 50 60 70 80 90 100 percent age p a s s i n g (%) silt sand gravel before shear after shear p'_0 (MPa) 0.2 2.0

Figure 21. Grain size distribution curves before and after shear.

the same figure. Open and full symbols identify peak and ultimate states, respectively; circles refer to

drained tests while squares refer to undrained tests. The dilatant behaviour observed in the undrained

tests results in the effective stress paths travelling to the right before the peak strength is attained. The peak strength points are fitted using a second order polynomial function, shown as a dashed line.

The tangent to the curve at $p' = 0$ yields a cohesion intercept $c' p = 0.3$ MPa and a peak friction angle

$\phi' = 45^\circ$. The end-of-test stress states fall closely around a straight line through the origin characterized

by a slope $M = 1.6$, or a friction angle $\phi' \sim 39^\circ$. Figure 20 shows the state paths followed during shear and the end-of-test states in a v - $\ln p'$ plot for

all intact samples and for one reconstituted sample (RIMAP) tested at high confining pressure. The

end-of-test points fall along a line which curves abruptly at a mean effective stress of about 3 MPa;

beyond this value of stress the state points describe a straight line, with a slope $\lambda = 0.132$. A bi-linear

or curved critical state line has been observed for a variety of sands (Been et al., 1991, Konrad, 1997,

Colliat-Dangus et al., 1988, Ishihara, 1993), and justified with grain crushing occurring during shear

at high effective stress, the change of slope in the critical state line reflecting changes in gradation and

grain shape which occur during shear. Some evidence of grain crushing was found examining the grain

size distribution curves of samples before and after shearing at increasing confining pressures, as shown

in Figure 21. p/p'_{cs} 0.0 0.0 0.5 1.0 1.5 2.0 2.5 3.0 3.5
4.0 1.0 2.0 3.0 q / p'_{cs} CID CIU end-of-test CS

Figure 22. Normalised stress paths for some drained and undrained triaxial tests on intact samples.

For the Pozzolana Nera the end-of-test conditions for dilatant samples may not be representative of

a true critical state. At low confining pressures, strain localization inhibits dilation in the sample, so

that critical state is only attained in a relatively narrow shear band where most rearranging and crushing

of grains, aggregates of grains and inter-particle bonds occur. Voids ratios are calculated from volume

changes measured at the boundary and referred to the whole sample and the volume strains occurring

within the shear band are grossly underestimated (Desrues et al., 1996). The arrows in Figure 20 indicate

the likely direction of state change for dilatant samples, although with the available information it is

not possible to quantify the amount by which the end-of-test state points should move upwards. In fact,

according to Lee & Coop (1995) the reported curvature of the critical state line of sands results from either

incomplete testing, as for soil states that are dry of critical the dilation required to reach a true critical

state may be incompatible with the apparatus, or from localization of strains at low confining pressures.

In the following, data are normalized with respect to the critical state line defined by the end-of-test

points of contractant samples, characterized by $\lambda = 1.88$ and $\lambda = 0.132$, and shown as a solid line in

Figure 20. With this choice of critical state line the ratio p' / p'_{cs} is constant and equal to 2.18 which is

close to that predicted by classic elasto-plastic models for soils (Schofield & Wroth, 1968, Roscoe &

Burland, 1968); Coop (1999) reported much larger spacing of the NCL and CSL for sands with values

of p' / p'_{cs} typically being around 4. Figure 22 shows the normalized stress paths for some drained and

undrained triaxial tests on intact samples; the critical state is identified by the crossed circle ($M = 1.6$,

$p' / p'_{cs} = 1$). The end-of-test points for dilatant samples fall along a line at $M = 1.6$, although they fail to

reach $p' / p'_{cs} = 1$ for the reasons outlined above, while the normalized stress paths of the two tests sheared

at constant p' at high pressures do not reach the same value of M at critical state. Consistently with data

for carbonate sands (Coop & Atkinson, 1993, Coop, 1999) the critical state or end-of-test points do not

correspond to the apex of the normalized stress paths.

Peak strength data were also normalized using the state parameter ψ (Been & Jefferies, 1985) which

accounts for the voids ratio and stress level at the start of shearing relative to the critical state line, as

defined in Figure 20. Samples characterized by negative values of ψ dilate during shear, while samples

with positive values of ψ compress. Figure 23 shows the peak friction angle measured in each test,

ϕ' / p' , versus the state parameter, ψ , together with a set of data for different sands (Been & Jefferies,

1985); ϕ'_{p} in this context refers to the angle of shearing obtained from the mobilized stress ratio at

peak as $\sin \phi'_{p} = 3\eta_{p} / (6 + \eta_{p})$. Although the peak friction angles obtained for intact Pozzolana Nera are

systematically higher than those reported by Been & Jefferies (1985) a similar decreasing trend of ϕ'_{p}

with ψ can be observed in Figure 23.

4.6 Stress dilatancy

Following Rowe (1962), for all drained and undrained triaxial tests the relationship between mobilised

stress ratio, $R = \sigma'_{a} / \sigma'_{r}$, and dilatancy, $D = 1 + d$, was examined, where -0.40 -0.30 -0.20 -0.10 0.00 0.10 0.20 ψ 30 40 50 60 70 ϕ_{p} (°) data obtained for sands (Been & Jefferies, 1985)

Figure 23. Effect of initial state on mobilized peak strength.

is the plastic strain increment ratio. Negative increments of plastic volume strain (plastic dilation)

correspond to $D > 1$ and positive increments of plastic volume strain (plastic contraction) correspond to

$D < 1$; at critical state ($\delta \epsilon_{p v} / \delta \epsilon_{p a} = 0$) $D = 1$. In Rowe's original formulation all strains are essentially plastic, or irrecoverable, as they result from

slip at the contacts of rigid particles. In the following, increments of plastic volume and axial strain were

calculated as:

The elastic axial strains, neglected in the analysis, are likely to be most significant at the beginning of

the shearing stage, which may affect the initial portion of the stress-dilatancy relationship. Equation (4)

simplifies for the case of undrained tests, in which $\delta \epsilon_{v} = 0$ and the voids ratio remains constant during

the test. Based on the study of the energy ratio at point

contacts of regular packings, Rowe (1962) suggested

that for a purely frictional granular material, the stress ratio, R , which represents the mobilised strength,

is proportional to the dilatancy, D , through a coefficient which depends solely on the angle of friction

between particles, ϕ_f :

The smallest value for ϕ_f is the angle of inter-particle friction on sliding, ϕ_μ , which depends on the

mineralogy, shape and roughness of the surface of the particles; the largest possible value of ϕ_f is

the angle of shear strength at large strains, ϕ'_{cs} , when the assembly of particles is being continually

remoulded and rearranged at constant volume. During shear the inter-particle angle of friction increases

monotonically from its minimum to its maximum value (Rowe, 1971). Therefore Rowe's stress-dilatancy

theory predicts that the ratio R/D during the test is always less than its final value which is attained at

large shear strains when the particles are continuously rearranging at constant volume and constant stress

(critical state). Many Authors have used Rowe's stress-dilatancy theory to interpret the mechanical behaviour of

natural stiff clays (Canestrari & Scarpelli, 1993, Rampello et al., 1993), coarse grained weak rocks

(Aversa, 1991) and very dense gravel (Flora & Modoni, 1998) in an attempt to identify the contributions

to strength due to friction and dilatancy and those due to microstructure and interparticle bonding. Figure 24 shows two typical stress-dilatancy relationships for drained triaxial tests carried out at

low confining pressures ($p'_0 = 0.05\text{--}0.70$ MPa). The small contractant volume strains at the beginning

of shear are reflected by values of $D < 1$; at a very early

stage of the test, however, the volume strain

increments become negative, as the soil dilates towards its ultimate state. As shearing progresses, the

0.0 0.5 1.0 1.5
2.0 2.5 $D = 1 + d$ 0 4 8 12 16 $R = \sigma'_a / \sigma'_r$ D_{max} (R/D)
 $\max R$ $\max R$ $\max D$ $\max (R/D)$ $\max p'_0$ (MPa) 0.20 0.35

Figure 24. Typical stress-dilatancy relationships for drained triaxial compression tests on intact samples at low

confining pressures. 0.00 0.25 0.50 0.75 1.00 1.25 $D = 1 + d$
0.00 1.25 2.50 3.75 5.00 6.25 7.50 $R = \sigma'_a / \sigma'_r$ D_{max}
(R/D) $\max R$ $\max p'_0$ (MPa) 2.00 2.50 R_{max} D_{max}

Figure 25. Typical stress-dilatancy relationships for drained triaxial compression tests on intact samples at

intermediate confining pressures.

stress-dilatancy paths are followed clockwise. Three characteristic points have been marked on each

stress-dilatancy curve in Figure 24. The first corresponds to the point in the test where the ratio R/D

attains its maximum value; as shearing progresses, the stress ratio, or mobilised strength, increases and

reaches its maximum value, R_{max} . The third characteristic point corresponds to the maximum dilatancy,

D_{max} . Figure 25 shows two typical stress-dilatancy relationships obtained at larger confining pressures

($p'_0 = 1.4$ -2.85 MPa); the same characteristic points as in Figure 24 are marked on the stress-dilatancy

curves in Figure 25; at these confining pressures the behaviour is contractant and the maximum dilatancy

is reached at the end of test ($D_{max} \sim 1$).

For both dilatant and contractant behaviour, the ratio $R/D = K$ increases and then decreases as shearing

progresses, attaining values in excess than $K(\phi'_{cs})$ and the peak state ($q_{max} = R_{max}$) is always reached at

shear strains smaller than those corresponding to the maximum dilatancy. 0.0 0.5 1.0 1.5 2.0 2.5 $D = 1 + d$ 0 4 8 12

16 $R = \sigma'_{a} / \sigma'_{r}$ C.C. M.C.C. p'_0 (MPa) 0.20 2.00
 Rowe (dilatant behaviour) Rowe (contractant behaviour)

Figure 26. Comparison of experimental data and theoretical stress-dilatancy relationships for dilatant and contractant

behaviour. Figure 26 shows the stress-dilatancy relationships proposed by Rowe (1962) for dilatant and contrac

tant behaviour together with the experimental data; in the same figure the stress-dilatancy relationships

predicted by two classic isotropic hardening elasto-plastic models for soils, namely Cam Clay (Schofield

& Wroth, 1968) and Modified Cam Clay (Roscoe & Burland, 1968), are also reported for comparison.

The theoretical relationships have been drawn assuming $\phi_{\mu} = 10^\circ$, $\phi'_{cs} = 39^\circ$ and $M = 1.6$. None of these

models are able to reproduce the experimental data; although the Cam Clay and Modified Cam Clay

flow rules can predict values of K during the test greater than the final value, the maximum value of R

is always attained at the maximum dilatancy, and no peak stress ratio can be predicted for contractant

behaviour ($D < 1$). Experimental data have been interpreted with a version of the stress-dilatancy theory (Rowe et al.,

1963) which accounts for the cohesion between particles, c_f :

Rowe et al. (1963) assumed that K depends on ϕ_f in the same manner as in Equation (6), while the

cohesion between particles is a material property. According to Equation (7), the ratio R/D can take

values in excess of $K(\phi'_{cs})$ depending on the cohesion between grains and the effective stress, σ'_{r} . The points where R/D is maximum may be thought of as states corresponding to the onset of a process

of "de-structuration" of the material involving the progressive reduction of inter-particle cohesion while

the angle of friction between particles, ϕ_f , increases from ϕ_μ to ϕ'_{cs} . The progress of these phenomena

is probably linked to both volume and shear plastic strains experienced by the material during the test.

The condition $\phi_f = \phi'_{cs}$ can be attained not only for shear strains smaller than those corresponding to

the critical states but even than those corresponding to the maximum dilatancy, which is attained at

relatively large shear strains. On the other hand, the shear strains at the maximum values of the ratio R/D

are relatively small, so that it is reasonable to assume that $\phi_f \approx \phi_\mu$ and that the mobilised shear strength

is mostly due to inter-particle cohesion. The peak deviator stress results from the interplay between

degradation of inter-particle bonds (corresponding to a reduction of c_f), increasing friction between

particles (increase of ϕ_f) and increasing rate of dilation (increase of D). At shear strains corresponding

to the point of maximum dilation (see Figures 24, 25), the interparticle friction has reached its maximum

value, $\phi_f \approx \phi'_{cs}$, while cohesion between grains seems to be completely lost ($c_f \approx 0$); the data can be

interpolated with a straight line through the origin at a constant slope $K(\phi'_{cs})$, making it possible to

evaluate the value of ϕ'_{cs} from the whole of the data on the last portion of the stress-dilatancy curve. To explore further this interpretation of the experimental data, in Figure 27 the values of $(R/D)_{max}$

obtained from drained and undrained triaxial compression tests on intact samples are plotted versus the

radial effective stress; for undrained tests this is calculated from the radial stress and the excess pore 0.01
 $0.1 \ 1 \ 10 \ 100 \ \sigma'_r \text{ (MPa)}$ $0 \ 20 \ 40 \ 60 \ 80 \ (R/D)_{max}$
 $K(\phi'_{cs}) \ K(\phi_\mu) \ K(\phi'_{cs}) \ K(\phi_\mu) \ c_f = 0.5 \text{ MPa}$ $c_f = 0.7 \text{ MPa}$ CID CIU

Figure 27. Effect of confining stress on maximum ratio R/D .
 1.0 1.5 2.0 2.5 3.0 3.5 D/p 30 40 50 60 70 ϕ/p (°) (Rowe, 1962)

Figure 28. Mobilized friction vs rate of dilation at peak from drained triaxial compression tests on intact samples.

pressure measured at $(R/D)_{max}$. In the same figure Equation (7) has been plotted for comparison using

two different constant values of $c_f = 0.5$ MPa and $c_f = 0.7$ MPa and constant values $\phi_f = \phi_\mu = 10^\circ$ and

$\phi_f = \phi'_{cs} = 39^\circ$. The trend of the data is in very good agreement with Equation (5), with $(R/D)_{max}$ reducing

with increasing σ'_r , although the data are not fitted exactly by any of the four curves in Figure 27. The

maximum values of (R/D) , however, were reached at different values of shear strain, so that it is likely

that the values of ϕ_f were not the same for all tests; quite significantly, the points relative to tests in

which $(R/D)_{max}$ was reached at lower shear strains tend to fall closer to the curves at $\phi_f = \phi_\mu$, while the

points from tests in which $(R/D)_{max}$ was reached at larger shear strains tend to fall closer to the curves

at $\phi_f = \phi'_{cs}$. However, the scatter in the data may also result from the natural variability of the material

which may produce different values of inter-particle cohesion c_f from one sample to another. The values

of inter-particle cohesion used to calculate the analytical curves are higher than the cohesion intercept

obtained from the failure peak envelope in $q-p'$; this is hardly surprising as the peak deviator stress is

always reached after the point in the test where R/D is maximum, which corresponds to the onset of the

de-structuration process.

Figure 28 shows the values of mobilised friction angle at

peak, $\phi' p$, versus the observed rate of dilation

at peak, D_p , which, for the Pozzolana Nera, is not the maximum rate of dilation, D_{max} . The experimental

data plot above the solid line which represents the expected relationship according to (Rowe, 1962),

indicating that there are extra components of strength, in addition to friction and dilatancy. The observed $-0.4 -0.3 -0.2 -0.1 0.0 0.1 0.2 \psi 0.0 0.5 1.0 1.5 2.0 2.5 3.0 3.5 D_p$

Figure 29. Effect of initial state on observed dilation at peak.

rate of dilation at peak, however, seems to be a reasonably well defined decreasing function of the state

parameter, ψ , as shown by the data in Figure 29; as expected, for $D_p = 1$, the state parameter ψ is null,

which confirms the choice of reference state line in $v:\ln p'$ plane. Figure 30(a), (b), and (c) shows the stress-dilatancy relationships obtained for three series of drained

shearing cycles which were carried out at increasing values of confining pressures ($p' = 0.35, 1.40$ and

2.00 MPa) before the main shearing stage of test BPHP; in each series of three cycles the maximum

deviator stress was kept about $1/10$ of the expected peak strength of the material at the corresponding

mean effective stress. The stress-dilatancy paths followed in each cycle are shown as continuous lines,

although the data are quite scattered. In the loading stage of each series, for any given value of R , D

increases with increasing number of cycles, which suggests that the structure of the material modifies

progressively for each subsequent cycle. The effect of the number of cycles reduces with increasing

confining pressure. Figure 31 shows a selection of the experimental stress-dilatancy relationships for a wide range of

confining pressures in terms of stress ratio, η , versus plastic strain increment ratio $d^* = \delta \epsilon_p v / \delta \epsilon_p s$. The

data corresponding to the points in the test where R/D is maximum are shown in the figure as crossed

squares and fitted using a straight line (line a) of equation:

where $\alpha = 1.49$ and $M^*(M) = 2.31$ is the mobilised stress ratio where the plastic volume strain

increment is null. A similar linear relationship fitting experimental data during de-structuration was observed, among

others, in Wood (1995) for a porous calcarenite. The continuous straight line (line b), at a slope $\alpha = 1$ and

intercept $M = 1.6$, fits closely the last portion of the observed stress-dilatancy relationships for dilatant

samples. From Equation (8), the mobilised stress ratio at the onset of de-structuration results from the

sum of a component due to friction and cohesion (M^*) and a component proportional to the dilatancy

(d^*). Lines a and b enclose the region of $\eta:d^*$ plane where the process of de-structuration takes place;

line b has the same equation as the Cam Clay (Schofield & Wroth, 1968) flow rule in which $M^* = M$

and $\alpha = 1$. Cotecchia & Chandler (1997) observed that the non-frictional character of the mechanical

behaviour results in values of α larger than unity for drained shearing.

4.7 Stiffness

The stiffness of intact Pozzolana Nera for axial strain in the range 0.001-0.25% was obtained from

triaxial compression tests at increasing values of confining pressure with intermediate cycles of loading

and unloading. A limited number of bender element tests was

also carried out during high pressure

Figure 30. Stress-dilatancy relationships observed in cycles of shear at constant radial effective stress;

(a) $p' = 0.35$ MPa; (b) $p' = 1.40$, and (c) $p' = 2.00$ MPa.

isotropic compression tests on natural samples and medium to low pressure isotropic compression tests

on reconstituted samples to examine the small strain ($<0.001\%$) stiffness of the material.

In the bender element tests the transmitter was excited using a single sinusoidal pulse with a peak-to

peak amplitude of 20V and a variable frequency (5-250 Hz). The arrival time for the received wave was

determined with the criteria suggested by Viggiani & Atkinson (1995). The quality of the received signal

decreased for the natural sample with increasing confining pressure, probably due to an increasing

mismatch of acoustic impedances between sample and transducer together with imperfect coupling

between the two. In some cases, in the initial portion of the received signal it is possible to identify the

arrival of a train of compression waves ($V_P \sim 2000$ m/s); in these cases, the arrival of the shear wave was

identified as a sudden change of the amplitude and frequency of the received signal (Sayers et al., 1990).

All the stress strain curves shown in Figure 16 show an approximately linear initial portion whose

average slope, $\delta q / \delta \epsilon_s$, in the range of shear strain $0.1 < \epsilon_s < 0.25\%$, does not depend on the confining

pressure, see Figure 32. The average value of the slope of the linear portion of the stress strain curve is

$\delta q / \delta \epsilon_s = 0.75$ MPa, while all data fall in the range $0.4 < \delta q / \delta \epsilon_s < 1.48$ GPa.

During some of the isotropic compression stages of the triaxial tests, axial loading-unloading cycles

were carried out in drained conditions at increasing confining stress; in these tests axial strain was

measured locally using two miniaturised LVDTs directly attached to the sample. The maximum value of

the deviator stress during each cycle, q_c , was well below the estimated value of the peak deviator stress

at the corresponding mean effective stress, q_{max} .

Figure 33(a) shows the loading unloading cycles carried out during test bphp ($q_{max}/q_c \sim 10$); even

at relatively small strain, the strain is not recovered on unloading, with cumulated strain at the end of -1.0 -0.5
 0.0 0.5 1.0 1.5 2.0 d* 0.0 0.5 1.0 1.5 2.0 2.5 3.0 η a b
 p'_0 (MPa) 0.20 0.35 2.00 2.50 2.85 onset of
 destructuration

Figure 31. Stress-dilatancy relationships for drained triaxial compression tests on intact samples. $0.1 \leq p'$
 (MPa) 0 0.4 0.8 1.2 1.6 2 $\delta q / \delta \epsilon_s$ (G P a) 1

Figure 32. Average slope of stress strain curves at $0.1 \leq \epsilon_s < 0.25\%$ at increasing confining pressures.

the first cycle of the order of 0.003-0.015%. Figure 33(b) shows the stiffness decay with shear strain

amplitude for each first cycle of four series, computed as the average of five values of tangent stiffness.

In the same figure the range of small strain stiffness measured in isotropic compression with bender

elements is also reported for comparison. The stiffness values obtained for successive cycles, at any

value of the shear strain, are about 10% higher than those obtained in the first cycle. Again the stiffness

of the material does not depend on the confining pressure. Figure 34 summarises some results on the very small strain stiffness of the material. Figure 34(a)

shows the state paths followed by the material during

isotropic compression while Figure 34(b) illustrates

the variation of small strain stiffness, G_0 , with mean effective stress, p' . For reconstituted samples, on first loading, the small strain stiffness increases with mean effective

stress:

with $p_r = 1$ MPa, $A = 282-324$ for $v = 1.2-1.05$, and $n = 0.63 =$ constant. The values of stiffness meas

ured during loading-unloading cycles are not included in the figure for clarity; they indicate smaller

Figure 33. Loading unloading cycles during test bphp. (a) stress strain curves; (b) stiffness decay with shear strain

amplitude. $0.1 \ 1 \ 10 \ 100 \ p' \text{ (MPa)}$ 1.3 1.5 1.7 1.9 2.1 2.3

v REM1 REM2 REM3 NAT NCL CSL (a) $0.01 \ 0.1 \ 1 \ 10 \ 100 \ p' \text{ (MPa)}$
 $0.01 \ 0.1 \ 1 \ 10 \ G_0 \text{ (GPa)}$ NAT REM1 ($e_0 = 1.249$) REM2 ($e_0 = 1.089$) REM3 ($e_0 = 1.050$) (b)

Figure 34. Small strain stiffness ($<0.001\%$). (a) compression curves; (b) dependence of G_0 on p' .

values of the stiffness exponent ($n \sim 0.5$). The stiffness of the intact material is significantly larger than

that of the reconstituted material and is independent of the mean effective stress at least up to $p' \sim 10$ MPa,

that is well beyond the yield mean effective stress, $p'_y \sim 6.6$ MPa. The higher stiffness of the natural soil

can only in part be explained in terms of smaller voids ratio, and must be attributed to the natural

microstructure of the soil. Similar results, showing that the small strain stiffness of three different soft

rocks decreases and the variation of G_0 with effective confining stress increases as the weathering process

progresses were obtained by Nishi et al. (1989).

5 UNSATURATED BEHAVIOUR

Pyroclastic deposits of the Colli Albani are generally

above the water table, which is normally very deep.

The mechanical properties of the unsaturated material and its response to any variations of the degree of

saturation are very important in geotechnical problems such as natural and artificial cuts instabilities,

sinkholes or roof collapses of underground cavities. Similarly to Roma, in southern Italy the city of Napoli was affected by even more intense explosive

volcanic activities throughout the centuries and the subsoil of the urban area mainly consists of pyroclastic

soils and rocks originated by the eruptions of the volcanic district of Campi Flegrei. Cohesionless

Pozzolanas generally overlay the Neapolitan Yellow Tuff and other pyroclastic tuffs or soft rocks (see

paper by Picarelli et al., to this conference). Even in this case, over large parts of the city, the pozzolanas are located above the groundwater table in

unsaturated conditions. Due to exceptional rainfalls or leaks from aqueducts and sewerage, the saturation

of pozzolanas occasionally causes slides in natural and artificial slopes and settlements in other parts

of the subsoil; the shallower layers of pozzolanas may occasionally suffer from landslides developing in

flowslides. Instability phenomena are triggered by meteoric events inducing noticeable variations of the

degree of saturation of the deposits. The mechanical behaviour of unsaturated Neapolitan pozzolanas has been extensively investigated

over the past 10 years (Nicotera, 1998, Aversa et al., 1998, Nicotera et al., 1999a, 1999b) while, until very

recently, no experimental observations were available for the pyroclastic deposits of the Colli Albani.

An experimental research on the mechanical properties of partially saturated Pozzolana Nera started a

few years ago at the University of Perugia and is still in progress (Cattoni, 2003, Cattoni et al., 2004,

Cecconi et al., 2005a, b). The main aim of these studies was to examine the effects of the transition

from unsaturated to saturated conditions on the mechanical properties of the materials and to verify their

relevance with respect to the observed instability phenomena.

5.1 Experimental programme

The laboratory testing programme on partially saturated Pozzolana Nera consisted of volume extractor

tests, oedometer/wetting tests, and triaxial compression tests at constant suction. In the following, some

of the main findings of this experimental work are outlined with particular reference to the hydraulic

properties of the soil, volumetric collapse on wetting, and shear strength of the reconstituted material. Soil-water retention curves for the Pozzolana Nera were evaluated using a standard volumetric pressure

plate device provided with a 200 kPa a.e.v. ceramic disk ($k = 3 \times 10^{-7}$ cm/s) located at its base. Suction

increments/decrements (Δs) were applied by increasing/decreasing the air pressure in the cylinder, while

the pore water pressure was kept constant at atmospheric pressure. The air pressure is regulated by two

pressure converters installed in series and measured by means of a digital precision manometer with an

accuracy of ± 0.07 kPa and a full range of 10 kPa. For each suction increment (or decrement) the volume

water removed (drying path) or adsorbed (wetting path) can be measured through a glass burette, after

equalization is reached. Typically the equalization stages required approximately 2-3 days for each

suction increment. Oedometer tests were carried out in conventional cells with a diameter varying between 35.7 and

71.4 mm. The material was first oven-dried, mixed with de-aired water and compacted in thin layers,

directly into the mould. Changes in initial water content were minimised by covering the cell with a

cellophane film. Wetting tests were performed at constant vertical stress, after primary settlement had

occurred ($dh/dt \leq 1.7 \times 10^{-5}$ mm/min). Triaxial compression tests were carried out using a servo-controlled automated system for unsaturated

soils associated to a Bishop & Wesley triaxial cell, designed to accommodate 50 mm diameter samples.

Suction is controlled by applying, controlling and measuring independently positive values of pore air

and pore water pressures (axis translation technique). The apparatus is fully described by Nicotera et al.

(1999) and Aversa & Nicotera (2002). Table 3 summarises the initial values of water content (w_0), voids ratio (e_0), degree of saturation (S_r)

of all the reconstituted samples used for the tests discussed in this section, while the initial grain size

distribution is shown in Figure 35. In this reconstituted state, the material can be classified as a silty

sand ($U = 3 \div 18$).

5.2 Hydraulic properties: soil-water retention curves

The volumetric pressure plate device allowed to assess the soil retention curve of partially saturated

Pozzolana Nera for values of suction smaller than the air entry value of the ceramic disk (200 kPa). Figure 36 shows the experimental soil-water retention curves obtained from the three tests in terms of

normalised volumetric water content ($\theta_w = \theta_w / \theta_{ws}$) as a function of suction. The air entry value of the soil

can be estimated from the well defined kink of the curve, by using the procedure proposed by Fredlund &

Xing (1994). All soil-water retention curves are characterised by a relatively narrow hysteretic domain;

the air entry value (~ 8.5 – 9 kPa) is approximately the same for all the samples, thus indicating that the

Table 3. Initial physical properties of reconstituted samples and test conditions. EV: volume extractor; OED:

oedometer; TX: triaxial compression.

Type Sample w_0 e_0 S_{r0} θ_{w0} s (kPa) p (1) net (kPa) σ_v wetting (kPa) $\theta_{e a}$ wetting (%)

EV EV03 0.34 0.90 1.00 0.47 - - - EV04 0.27 0.73 1.00 0.43 - - - EV05 0.31 0.83 1.00 0.45 - - -

OED ED001 0.14 1.25 0.30 0.17 - - 100 (2) 0.094 ED002 0.13 1.13 0.32 0.17 - - 400 (2) 0.215 ED003 0.14 1.03 0.36 0.18 - - 1600 (2) 0.149 ED004 0.13 1.01 0.35 0.18 - - 800 (2) 0.400 ED005 0.10 1.15 0.23 0.13 - - 3200 (2) 1.079 ED006 0.13 1.60 0.21 0.13 - - 400 (3) 1.837 ED007 0.13 1.62 0.21 0.13 - - 400 (2) 1.022 ED008 0.14 1.45 0.25 0.15 - - 400 (3) 1.801 ED009 0.13 1.50 0.24 0.14 - - 400 (2) -0.025 ED010 0.13 1.60 0.21 0.13 - - 50 (4) 0.094

TX TX03 0.23 0.63 1.00 0.39 20 200 - - TX06 0.25 0.68 1.00 0.40 20 400 - - TX07 0.26 0.70 1.00 0.41 20 100 - - TX10 0.25 0.68 1.00 0.41 75 100 - - TX11 0.24 0.63 1.00 0.39 75 50 - - TX12 0.24 0.65 1.00 0.40 20 50 - - TX13 0.26 0.70 1.00 0.41 75 200 - - TX14 0.22 0.60 1.00 0.38 - 200 - - TX15 0.26 0.69 1.00 0.41 75 100 (5) - - TXMC 0.39 1.05 1.00 0.51 - 200 - -

(1) at the end of isotropic compression; (2) wetting on loading; (3) wetting after unloading-reloading cycle;

(4) wetting after unloading; (5) suction applied after isotropic compression

Figure 35. Grain size distribution of reconstituted samples of Pozzolana Nera.

Figure 36. Soil water retention curves for reconstituted Pozzolana Nera.

effect of initial grain size distribution overwhelms the

difference in initial voids ratio. In fact, the air

entry value is directly controlled by the maximum grain diameter, d_{max} , which is almost identical for the

three samples, while small differences in fine content result in different wetting-drying paths. Several equations for the soil-water retention curves are available in the literature. Among these, the

equation proposed by Fredlund & Xing (1994) and the one proposed by Van Genuchten (1980) have

been chosen to fit the experimental data, by using a non-linear curve fitting algorithm. The equation

proposed by Fredlund & Xing (1994) is:

with $e^N = \exp(1)$. In Equation (10) parameters a , m and n are defined as follows: parameter m depends

on the shape of the soil water retention curve close to the inflection point; a corresponds to the inflection

point of the curve, and for small values of m , is equal to the air entry value; parameter n depends on

the shape of the pore size distribution and increases with increasing the uniformity of the pore size

distribution. The equation proposed by Van Genuchten is: The quantity θ_{ws} is the volumetric water content in saturated conditions, while θ_{wr} is the residual

volumetric water content which has been estimated as the value of θ_w at $s = 150$ kPa. Similarly, three parameters a' , m' , and n' compare in Equation (11) but only two of them are inde

pendent since $m' = 1 - 1/n'$ (Mualem, 1976). Parameters a' and n' have the same physical meanings of

a and n in Equation (10). In Figures 37(a) and (b) the experimental drying paths are fitted using Equations (10) and (11). The

equation proposed by Fredlund & Xing fits nicely the experimental data in the whole range of suctions,

while Equation (11) seems to underestimate the experimental

data for suctions larger than 60 kPa. The

two sets of parameters entering Equations (10) and (11) are reported in Table 4.

5.3 Hydraulic properties: conductivity

Three different approaches have been used to evaluate the conductivity of Pozzolana Nera:

a) analytical methods based on the solution of Richards' equation for water flow in unsaturated soils (Gardner, 1956, Rijtema, 1959):

b) semi-empirical methods which make use of the measured soil water characteristic curve (Kunze et al., 1968, Green & Corey, 1971, Nielsen et al., 1972);

Figure 37. Experimental data and curve fittings; (a) Fredlund & Xing model; (b) Van Genuchten model.

Table 4. Curve fitting parameters and measured air entry values. Fredlund & Xing (1994) Van Genuchten (1980) a.e.v

Sample (kPa)	a (kPa)	m	n	a' (m ⁻¹)	n'	θ _{wr}
ev03rec	9.0	11.90	0.43	2.81	0.6	2.030
ev04rec	9.0	11.40	0.54	2.51	0.7	2.030
ev05rec	8.5	10.85	0.54	3.65	0.7	2.310

c) statistical methods based on the pore size distribution (Burdine, 1953, Mualem, 1976, Van Genuchten,

1980, Fredlund & Xing, 1994).

The methods belonging to the first class allow to estimate the conductivity function from the com

parison between the experimental data - namely θ_w vs. time - obtained during the transient equalisation

stages (pressure plate outflow data) and the theoretical solution derived by the integration of Richards

differential equation, at given initial and boundary conditions. In these methods it is assumed that, for

small increments of suction, both k_w (θ_w) and the

diffusivity coefficient, $D_w(\theta_w)$, may be taken constant. Therefore, the equation governing the one-dimensional water flow induced by a change in suction, becomes formally identical to the equation of 1D consolidation (Terzaghi, 1923). In particular, the conductivity function of unsaturated Pozzolana Nera has been determined by using the methods proposed by Gardner (1956) and Rijtema (1959); these differ in that the latter takes into account the impedance of the high air entry value of the ceramic disk.

In the second approach the experimental water retention curve is physically linked to a random pore size distribution. The conductivity function is given by a series obtained from the probability function

of interconnections between water-filled pores of varying sizes:

where:

A_d : adjusting constant depending on the surface tension of water, the absolute water viscosity and the

water density. It is assumed $A_d = 1 \text{ (m} \times \text{s}^{-1} \times \text{kPa}^2 \text{)}$;

k_{sc} : computed saturated coefficient of permeability from Equation (13) when $k_w(\theta_w)_i = k_s$;

k_s : measured saturated coefficient of permeability ($k_s = 10^{-7} \text{ m/s}$).

Finally, the third approach for the prediction of $k_w(\theta_w)$ - see point c) above - involves the methods

based on a reasonable approximate evaluation of the hydraulic conductivity of a pore domain with varying

shape (e.g.: Mualem, 1976). The conductivity function $k_w(\theta_w)$ is derived in a integral form and can be

Figure 38. Conductivity as a function of suction (a) and degree of saturation (b).

reduced to a closed form when a $\theta - w - s$ dependence is given by analytical expressions, such as the one

proposed by Van Genuchten (1980): In Equation (14), h is the suction potential ($h = s/\gamma_w$) and a' , m' , n' are the same parameters defined

in Equation (11). In Figures 38(a) and (b) the values of $k_w(s)$ calculated using the analytical solutions of Gardner (1956)

and Rijtema (1959) are compared with the values predicted by Equations (13) and (14) and with the

experimental data. The comparison shows that:

- the saturated coefficient of permeability k_s obtained with the analytical solution of Rijtema (1959) is in good agreement with the measured values;
- k_w decreases of about 4 orders of magnitude in the range of suction 0–100 kPa ($S_r = 1 \div 0.3$), independently of the adopted method;
- the values of the conductivity coefficient obtained with the solution of Rijtema (1959) are in good agreement with those calculated with the method proposed by Green & Corey (1971), but the data are quite scattered at small suctions;
- the k_w values derived from the solution proposed by Gardner (1956) are less accurate, so indicating the important effect of the impedance of the high air entry value of the ceramic disk;
- at large values of suction, the values of k_w computed with Equation (14) are sensibly smaller than those determined with the other two methods.

5.4 Volumetric collapse on wetting

Figure 39 shows the compressibility curves of four wetting tests performed in oedometer cells on par-

tially saturated samples characterised by different initial voids ratios ($e_0 = 1.0 \div 1.6$), similar degree of

saturations ($S_r = 0.2 \div 0.3$), and same initial water content ($w_0 \approx 0.13$). During loading the samples were submerged in distilled water and saturated at constant values of

vertical stress, σ'_{v_0} . The majority of the samples exhibited a “collapsible” behaviour on saturation with

increasing axial strain at constant vertical effective stress; the amount of axial strain due to saturation

depended on initial voids ratio, degree of saturation and stress level. In particular, for the initially denser

samples ($e_0 = 1.01 \div 1.25$), the larger the vertical stress the larger the axial strain due to saturation; no

significant collapse-strains are observed for $\sigma'_{v_0} < 3200$ kPa. On the other hand, looser samples ($e_0 = 1.45 \div 1.60$), which were also characterised by smaller degrees

of saturation, showed collapsible behaviour even at low stress levels ($\sigma'_{v_0} = 400$ kPa). Therefore, the effect

of the initial voids ratio and degree of saturation seems larger than that of the vertical stress (tests ED05

and ED07 in Figure 39 and Table 3).

Figure 39. Oedometer tests with wetting paths at different vertical stress and voids ratios.

Figure 40. One dimensional compression with wetting paths: a) compressibility curves; b) changes in voids ratio

induced by loading ($\sigma'_{v_0} = 400$ kPa) and following wetting.

The influence of having experienced unloading paths before wetting is shown in Figures 40(a) and

(b): samples saturated on first loading exhibit larger axial strains than those shown by samples which

underwent an unloading-reloading cycle before wetting (tests ED06 and ED07, see Table 3). Figure 40(b)

shows the ϵ -time curves induced by saturation for two couples of tests linked by the same load history

but different initial porosity. Other tests show that if wetting occurs along an unloading path, or a

reloading path well before reaching the normal compression line (1D-NCL), axial strains induced by

wetting may also be negative (swelling, see test ED010 after unloading). The same behaviour has been

observed on rockfill by Oldecop & Alonso (2001).

5.5 Stress-strain behaviour and shear strength

Figure 41(a) shows the experimental stress-strain curves obtained from three drained triaxial compression

tests carried out at constant suction $s = 20$ kPa, in terms of deviator stress, q , versus deviatoric strain,

$\epsilon_s = 2/3 (\epsilon_a - \epsilon_r)$; the corresponding curves of volume strains, $\epsilon_v = \epsilon_a + 2 \epsilon_r$, versus ϵ_s are shown in

Figure 41. Triaxial compression tests at constant suctions $s = 20$ kPa and $s = 75$ kPa: a) and c) stress-strain curves;

b) and d) volume strains vs. deviatoric strains.

Figure 42. Stress-strain curves from triaxial compression tests at increasing suction: (a) $p_{net,0} = 50$ kPa; curves;

(b) $p_{net,0} = 100$ kPa.

Figure 41(b). Similarly, the stress-strain curves obtained from other four triaxial tests at constant suction

$s = 75$ kPa are plotted in Figures 41(c) and (d). For both series of data, it can be noted that both peak deviator stress and initial stiffness increase

with the mean net pressure, p_{net} , as expected, while the deviatoric strain at peak is approximately the

same for all tests. The soil behaviour is dilatant and the rate of dilation (dev/des) slightly decreases

with increasing p_{net} , as common for granular soils. The maximum rate of dilation is generally achieved

before peak unlike what observed for saturated natural Pozzolana Nera, see Sections 4.4 and 4.6 above.

After peak, the deviator stress decreases towards an ultimate value (eot state). Volume strains after peak

are not reliable and have not been plotted in Figure 41,

because of strain localisation occurring along

shear bands. Figure 42 shows the results of tests performed at suctions of 20 and 75 kPa, after isotropic compression

to a mean net pressure $p_{net,0} = 50$ and 100 kPa. The data show that the shear strength increases as the

suction increases from 20 to 75 kPa; this increase of strength is up to 50% for the two tests at a mean

net pressure $p_{net} = 50$ kPa. Although samples TX10 and TX15 were sheared under the same suction

$s = 75$ kPa, the peak deviator stress attained by samples TX15 is significantly larger than that obtained

for sample TX10. This difference can be attributed to the different paths followed before shearing as

Figure 43. Triaxial compression tests on partially saturated and saturated samples): a) stress-strain curves; b) volume

strain vs. axial strain.

sample TX15 had been first isotropically compressed in saturated conditions, after which suction was

increased (see Table 3).

The results of two tests carried out on reconstituted saturated samples (TXMC and TX14), isotropically

consolidated to $p' = 200$ kPa are compared with the results obtained for unsaturated samples in Figure 43,

in terms of deviator stress vs. axial strain (Figure 43a) and volume strain vs. axial strain (Figure 43b). The

data show that: (i) the peak shear strength and the initial tangent stiffness both increase with increasing

suction; (ii) there are no significant effects of suction (or degree of saturation) on the ultimate shear

strength; (iii) the end-of-test condition does not depend on initial voids ratio, as expected.

For all samples showing dilatant behaviour, failure was

associated with very well defined shear

surfaces separating the sample in two parts.
Strain-localization may have occurred quite soon after peak;

therefore the stress-strain data at strains larger than those corresponding to peak conditions must be

regarded with caution and data corresponding to values of shear strains larger than about 7% have been

intentionally omitted in Figures 41 and 43.

The shear strength of the material was examined in terms of stress parameters s , q , p_{net} . The experi

mental results at peak have been interpreted with the Mohr - Coulomb failure criterion, extended to

unsaturated soils by Fredlund & Rahardjo (1993). The failure envelope is defined by the following

equation:

The failure envelope intercept q_0 is related to the total cohesion through:

where the total cohesion c results from two contributions, namely the effective cohesion, c' , and the

apparent cohesion:

where the friction angle ϕ_b represents the increase in cohesive strength due to suction.

It follows that the failure envelope in terms of stress invariants q , p_{net} can be rewritten as:

where $\tan \psi_b = \tan \phi_b / \tan \phi$ and $q_1 = c' h \cot \phi$.

Figure 44 shows the stress states at peak deviator stress and at the end of test (eot). Open and full

circles identify tests at values of suction of 20 and 75 kPa respectively, while cross symbols refer to the

stress states at the end of test.

The analysis of data leads to the shear strength parameters

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Characterisation and Engineering Properties of Natural
Soils - Tan, Phoon, Hight & Leroueil (eds) © 2007 Taylor &
Francis Group, London, ISBN 978-0-415-42691-6

Geotechnical characteristics of a pumice sand

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ABSTRACT: Extensive deposits of pumice sand are found in various parts of the North Island of New

Zealand, and their soft-grained nature raises important questions about their engineering properties,

especially during penetrometer tests. This paper describes an investigation of the properties of such a

sand. The prime aim was to investigate the behaviour of the sand during static cone penetrometer (CPT)

testing. For this purpose, a series of tests was carried out using a calibration chamber. In addition to the

chamber testing, a range of laboratory tests was carried out including compressibility, triaxial behaviour,

particle crushing, and large strain behaviour. For comparison purposes, many of the tests were duplicated

using a "normal" hard grained sand. This allowed direct comparisons to be made between the behaviour

of a soft and a hard-grained sand.

1 INTRODUCTION

This paper arises out of a study of a pumice sand found in the North Island of New Zealand. The study,

as originally conceived, had a specific aim, namely to investigate the way the sand behaved during cone

penetrometer tests. This was done by conducting a series of standard cone penetrometer tests (CPTs) in

a calibration chamber. The intention was to develop correlations between cone resistance and relative

density similar to those in existence for "normal" hard grained sands. Standard laboratory tests were also

planned to provide background information for the interpretation of the calibration tests. As the study

proceeded, additional testing was added to the programme to investigate other properties of relevance

and interest, in particular the crushing characteristics of the material, and behaviour at large strains in

triaxial testing. The study has been a collaborative effort by geotechnical staff in the Civil Engineering

Department at Auckland University, and most of what is presented here has been published in two earlier

papers, Wesley et al (1999), Wesley (1999).

2 ENGINEERING GEOLOGY

Sands consisting predominantly of pumice particles are found in various parts of the North Island of New

Zealand. They originate from a series of volcanic eruptions centred in the Taupo and Rotorua regions

of the central North Island. The last major eruption was approximately 2000 years ago. The pumice

material has been distributed initially by the explosive power of the eruptions and associated airborne

transport; this has been followed by erosion and river transport. The pumice deposits exist mainly as

deep sand layers in river valleys and flood plains, but may also be found as coarse gravel deposits in hilly

areas. The sand deposits are concentrated along the lower reaches of the Waikato River (the main river

flowing from the central Taupo volcanic plateau to the coast), and in the Bay of Plenty, where a number

of rivers have deposited pumice sands in their flood plains near the coast.

Although pumice sands do not cover wide areas, their concentration in river valleys and flood

plains means they tend to coincide with areas of considerable human activity and development. Conse

quently they are not infrequently encountered in engineering projects and their evaluation is a matter of

considerable interest to geotechnical engineers.

Figure 1. Pumice sand particle (approximate longitudinal dimension 2 mm).

3 COMPOSITION

Pumice sand is characterised by the vesicular nature of its particles; each particle contains a dense

network of fine holes, some of which are inter-connected and open to the surface, while others appear to

be entirely isolated inside the particles. The result is that the particles are light-weight, have very rough

surfaces, and are easily crushed, especially when compared to more "normal" hard grained sands such as

quartz sand. Finger-nail pressure on sand sized particles on a glass plate is normally sufficient to crush

them. The particle material is almost entirely quartz.

Figure 1 shows a photograph of a pumice particle.

4 STRUCTURE

As far as the author is aware, these sand deposits in situ have no particular structure. There is no

information to suggest that they are cemented in any way. An earthquake in the Bay of Plenty in the

1987 caused liquefaction of the sand in a number of places, which appears to confirm their uncemented

nature.

5 STATE AND INDEX PROPERTIES

5.1 Basic properties

To allow direct comparison of the pumice sand behaviour with that of a hard grained sand, many of the

tests were duplicated, with identical tests being carried out on both the pumice sand and on a hard grained

sand. The two sands were obtained from sand processing plants that dredge sand from the Waikato River.

Although they are natural sands in the sense that they occur naturally in the river, they have both been

processed by the dredging plants, the pumice sand having been separated out from the quartz sand

dredged with it, and the quartz sand processed to remove any soft grains from it. The sands were selected

to have particle size distributions as similar as possible; these are shown in Figure 2. Conventional tests on both sands gave maximum and minimum density values and void ratios as

shown in Table 1. These parameters do not appear problematical, but this is not the case with the void

ratio of the pumice sand. The void ratio is calculated from the specific gravity, and measurement of the

specific gravity of pumice sand is problematical, as explained in the following section.

5.2 Specific gravity

Pumice sand particles, in effect, have two densities, and thus two specific gravities. Firstly, there is the

density of the particle material itself, in this case quartz, which can be expected to have a specific gravity 2.65. Particle size mm Sieve size mm μ m 100 80 60 40 20 0 Percentage finer Pumice sand Quartz sand 61 10.1 150 300 600 1.18 2.36

Figure 2. Particle size curves of the two sands. Table 1. Density and void ratios of the two sands. Pumice Quartz Dry density Max 730 1520 (kg/m³) Min 620 1320 Void ratio Min 1.42 0.71 Max 1.85 0.97

of about 2.7. Secondly, there is the density of the whole particle, which consists partly of solid material

and partly of hollow space. Large pumice particles (greater than about 2 mm) float in water, so they

clearly have a specific gravity less than one.

If the specific gravity is measured in the standard way using vacuum extraction to remove air, it will

almost certainly mean that some air is removed from the holes inside the particles as well as from the

void space between the particles. The measured particle volume will then be underestimated and the

specific gravity overestimated. Even if vacuum extraction is not used, surface tension effects will almost

certainly “suck” some water into the internal void space.

To investigate this issue, and to arrive at an appropriate value of specific gravity for use in void ratio

calculations, a series of specific gravity tests was carried out on a range of pumice samples consisting of

fractions of different sizes in addition to the natural sand. The material was crushed and sieved to obtain

these fractions. The tests were done in two ways. Firstly, a simple displacement technique was used

without air extraction, and secondly the standard procedure was used. The belief was that the first tests

would minimise penetration of water into the internal voids and would thus give the particle specific

gravity, while the second tests would maximise penetration into the internal voids and would give specific

gravity values approaching that of the material of which the pumice was composed, assumed to be quartz.

The results are shown in Figure 3.

It is seen that there is a substantial difference between the two values for all particle sizes, as would

be expected, and that both values steadily increase as the particle size decreases. It appears that as the 2.5 2.0 1.5 1.0 0.5 Particle size (mm) A p p a r e n t s p e c i f i c g r a v i t y Standard procedure (with vacuum extraction) “Direct” procedure (simple displacement) Values from natural sand 0.04 0.1 1.0 10 20

Figure 3. Specific gravity measurements, using simple displacement, and vacuum extraction, on pumice sand

fractions of varying size.

particle size is reduced the proportion of the internal voids into which water penetrates during the tests

increases and the measured specific gravity increases. It seems surprising that even with the very fine

fraction the specific gravity value does not approach that of quartz, so that even the very fine particles

contain internal voids into which water cannot penetrate. The tests confirm that the internal voids in the

particles are not all interconnected. However, the tests do not provide any clear basis on which to decide

an appropriate specific gravity. The value adopted for the pumice sand was 1.77, this being the value

obtained without vacuum extraction. It is probable that this is an overestimate of the true value.

5.3 Compressive strength

An indication of the strength of the pumice grains has been obtained by carrying out unconfined com

pression tests on several samples. These samples were cubical in shape about 2 cm in length and 1 cm in

width. They were trimmed from gravel sized pieces of pumice believed to be of similar composition to

the grains of which the sand was composed. The average value of unconfined compressive strength was

2.7 MPa. This is very low compared with the values expected from hard materials such as quartz, which

would normally be in the order of 50 to 100 MPa.

6 ENGINEERING PROPERTIES

6.1 General

All laboratory tests have been carried out on dry samples. It is not practical to carry out large chamber

tests in any other way, and for uniformity the same procedure has been adopted for the laboratory tests.

Several tests were conducted to assess possible differences between the behaviour of the material when

dry with that when wet. These are described later and show that wetting the sand had a measurable, but

very small influence on its behaviour.

6.2 Compressibility characteristics

6.2.1 Standard oedometer test

The comparative compressibility of the sands in their loose and dense states was first measured using

conventional oedometer tests. The results are given Figure 4 and clearly show the pumice sand to be

much more compressible than the quartz sand. Up to a stress of about 2000 kPa it is about four times as

compressible in both the loose and dense states. Stress (kPa) 0 1000

2000 10 20 C o m p r e s s i o n (%) Q U A R T Z P U M I C E Dense Loose Dense Loose

Figure 4. Compressibility measured in standard oedometer tests.

6.2.2 High pressure oedometer tests and measurements of particle crushing

To investigate the extent of crushing that occurs in the pumice sand at different stress levels, a series of

one-dimensional compression tests was carried out using samples of varying particle size. Four different

samples were tested - the "natural" sand already described plus three samples of pure pumice. These

were obtained by crushing large pieces of pumice and then sieving the crushed material into fractions

of known particle size. The fractions used were:

- 1) 4.0 mm to 9.5 mm

2) 1.18 mm to 600 μm

3) 150 μm to 75 μm

These thus covered fine gravel, medium to coarse sand, and very fine sand. The “natural” sand, used

in the penetrometer testing, was a medium to coarse sand. The nature of these sieved fractions is rather

different to that of the “natural” sand because they consist of crushed fragments. The “natural” sand

was an alluvial deposit and its formation and transportation process resulted in predominantly rounded

particles. The crushed fractions, on the other hand, consisted of very angular particles. This difference

was expected to have an influence on their behaviour in the compression tests. The particle sizes curves

are illustrated in Figure 5.

Samples were prepared in the loose and dense states and loaded in stages to the following stress levels:

500 kPa, 1500 kPa, 3000 kPa, 5000 kPa, and 8000 kPa.

These values were selected to cover the range up to the maximum stress involved in the penetrometer

tests described later in this paper, which was about 8 MPa. The sample size was identical to that used

in conventional consolidation tests, namely 76 mm in diameter and 19 mm in thickness. To apply the

high stresses involved, the loading method was by means of a mechanical jack and a loading frame.

Measurements of the compression of the sand were made as the load was applied. When each of the

stress levels above was reached, the samples were unloaded and the material removed and a conventional

sieve analysis carried out.

The results of these tests, given in Figures 6 and 7 show following significant features:

1) The initial parts of the curves are close to linear (as indicated earlier in Figure 4), but the material

becomes gradually stiffer as the pressure increases.

2) The natural pumice sand is the least compressible of the pumice sands, although the finest of the

sieved fractions is only slightly more compressible. The compressibility of the two coarser fractions

is very much greater, especially at low stress levels. 0.05
0.1 1

10 Particle Size (mm) 75 150 300

600 1.18 2.36 4.0 9.5

100 80 60 40 20 Percent Passing Natural
pumice sand Crushed pumice sands (μm)

(mm) Sieve Size

Figure 5. Sands used for particle crushing measurements in high pressure oedometer tests. 10 20 30 40 50 60 Pressure (kPa) Compression (%) 4.0 - 9.5 mm 600 μm - 1.18 mm 75 μm - 15 μm Natural Quartz sand LOOSE SAMPLES 0 2000 4000 6000 8000

Figure 6. Compressibility of loose pumice samples in high pressure oedometer tests.

3) At high stress levels the curves are tending to run parallel to each other, indicating that compression has probably resulted in particle crushing and the creation of materials that are reasonably well graded and of similar stiffness. The data in Figures 6 and 7 is shown also in Figure 8 as void ratio versus log (pressure) graphs. Two

graphs, representing the loose and dense states, are shown for each material. The void ratio in Figure 8 has been calculated using a specific gravity value of 1.77 for all samples.

This is a reasonably representative value for all four samples, but is not necessarily the true value, and

the calculated void ratios should not be regarded as absolute values. They are useful for comparative

purposes only. Figure 8 confirms the observation made above

that at high stress levels the curves for

each material tend to approach and run parallel to each other, indicating that the compressibility of the

samples becomes fairly similar. It is important to recognise that despite the shape of the curves in Figure 8, none of these compression

curves from the pumice sand samples indicate a "yield" pressure. The linear plots in Figures 6 and 7 are

the only reliable indicators of a yield pressure and these confirm the absence of any such pressure. 10 20 30 40 50 60
Pressure (kPa) C o m p r e s s i o n (%) 4.0 -
9.5 mm 600 μ m - 4.0 mm 75 μ m - 150 μ m
Natural Q u a r t z s a n d DENSE SAMPLES 0 2000 4000 6000
8000

Figure 7. Compressibility of dense pumice samples in high
pressure oedometer tests. 4 3 2 1 0 V o i d r a t i o 100
1000 10000 Pressure (kPa)
4.0 - 9.5 mm 600 μ m - 4.0 mm 75 μ m - 150 μ m
Natural Loose Dense

Figure 8. Conventional void ratio versus log pressure
curves for the pumice sands.

The results of the particle size measurements made after
each test are shown in Figure 9. These results

are all from the loose samples; it was found that while the
degree of crushing was greater with the loose

samples than with the dense samples the difference was
quite small.

These results show the following important points:

1) The extent of particle crushing is very large. With the
coarsest fraction, the proportion passing the

4.0 mm sieve at the highest stress level of 8000 kPa has
risen from zero prior to testing to almost 90

percent. 0.05 0.1
1 10 Particle
Size (mm) 75 150 300 600 1.18
2.36 4.0 9.5 (μ m) Sieve Size
(mm) (μ m) Sieve Size

(mm)

100

80 60 40 20

P e

r c

e n

t P

a s

s i

n g Natural sand Initial 500 kPa 1500 kPa 3000 kPa

5000 kPa 8000 kPa. 0.05 0.1

1 10 Particle

Size (mm) 75 150 300 600 1.18

2.36 4.0 9.5 (µm) Sieve

Size (mm) Initial 500 kPa 1500

kPa 3000 kPa 5000 kPa 8000 kPa. 100 80 60 40 20 P e r c e

n t P a s s i n g Fraction 600 µm - 1.18 mm 0.05 0.1

1 10 Particle

Size (mm) 75 150 300 600 1.18

2.36 4.0 9.5 Initial 500 kPa 1500

kPa 3000 kPa 5000 kPa 8000 kPa. 100 80 60 40 20

P e

r c

e n

t P

a s

s i

n g Fraction 4 mm - 9.5 mm 0.05 0.1

1 10 Particle

Size (mm) 75 150 300 600 1.18

2.36 4.0 9.5 (µm) Sieve Size

(mm) 100 80 60 40 20 P e r c e n t P a s s i n g

Initial 500 kPa 1500 kPa 3000 kPa 5000 kPa 8000 kPa.

Fraction 75 µm - 150 µm

Figure 9. Particle size measurements at each stress level for the four sands tested.

2) The crushing is most pronounced with the coarsest material, ie the 4.0 mm to 9.5 mm fraction.

3) The extent of crushing with the "natural" sand at the highest stress level is very similar to that measured in the chamber penetrometer tests, described in a later section of this paper. Figure 10 illustrates the relationship between degree of crushing and stress level. The measure used

here for degree of crushing is a fairly elementary one, and alternative, perhaps more sophisticated,

measures can be found in the literature. The measure used here has been calculated by simply comparing

the amounts retained on each sieve; where reductions have occurred these have been added up to give

the total weight of material crushed. This has then been expressed as a percentage of the total weight. The main points to come out of these tests are the following:

1) With two of the samples, (4-9.5 mm and 600 μ m-1.18 mm) crushing starts at very low stress levels. There is already significant crushing when the first measurement is made at a stress of 500 kPa. With the other two samples, the natural sand and the fine fraction (75 μ m-150 μ m), there is negligible crushing until about 1000 kPa. An attempt was made to better define the stress at the onset of crushing, but measurements at low stress levels with only a very small amount of crushing were not found to give repeatable results and were not pursued further.

2) With all the samples, most of the crushing occurs between a stress of about 1000 kPa and 3000 kPa (the steepest part of the curves). Examination of the stress deformation curves in Figures 6 and 7 shows that this is the region where the samples are all stiffening up. Thus the onset of particle crushing does not result in any increase in compressibility of the material. If anything it marks the stress level at which the material begins to stiffen up.

3) While particle crushing at stress levels below 1000 kPa was not large enough to be measured reliably, it is

believed that minor crushing still occurred and was responsible for the much higher compressibility of the pumice samples illustrated in Figures 4, 6, and 7.

4.0 - 9.5 mm 600 μ m - 4.0 mm 75 μ m - 150 μ m
 Natural Pressure (kPa) 100 80 60 40 20 D e g r e e o f C
 r u s h i n g (%) LOOSE SAMPLES (a) Loose samples
 4.0 - 9.5 mm 600 μ m - 1.18 mm 75 μ m - 150 μ m
 Natural Pressure (kPa) 100 80 60 40 20 D e g r e e o f C
 r u s h i n g (%) DENSE SAMPLES (b) Dense samples 0 0
 2000 4000 6000 8000 2000 4000 6000 8000

Figure 10. Degree of crushing in high pressure oedometer tests.

6.2.3 Influence of wetting

As mentioned earlier, all tests were carried out on dry sand. To determine whether the results would be

the same if the sand was wet, some limited oedometer testing was undertaken on both dry and submerged

samples. Several tests were done in each case on samples in the loose and dense state. The results are

shown in Figure 11. This shows that there is a small increase in compressibility with submergence, but

not sufficient to significantly affect general behaviour.

6.3 Triaxial tests

The triaxial tests were performed using free end platens on samples of a nominal diameter and length of

70 mm. The intention with this procedure was to induce uniform strains and to investigate behaviour at

large strains. The oversized "free end" platens had a diameter 15% greater than the sample diameter, and

two layers of thin rubber sheeting with silicone grease between them were placed between the sample

and end platens. Small porous stones were inset at the centre of the platens to ensure pore air pressure

remained at atmospheric. All tests were thus "drained" on dry samples of the natural pumice sand. The

loose samples were prepared by pouring sand through a funnel and the dense samples were prepared by

Figure 11. Influence of wetting on compressibility.

vibration. Typical behaviour during triaxial testing is illustrated in Figures 12 and 13, which give results

from loose and dense samples at low (50 kPa) and high (300 kPa) confining stresses. The most significant characteristics from these tests are the following:

- a) At the low stress level of 50 kPa, the behaviour of the two sands is fairly similar. In the dense state both materials show clear peak values of deviator stress, followed by a decrease and levelling off towards critical state values at strains of 20% to 30%. In the loose state peak values are still evident but much less distinct than with the dense samples. Volume change is more or less as expected, with all samples showing some dilation during shearing.
- b) At the high stress level of 300 kPa, the quartz sand continues to behave as expected, but the pumice sand no longer shows clear peak values of deviator stress. Also the pumice sand no longer shows any dilatant behaviour, in either the loose or dense state.
- c) The strains to reach peak strength are significantly larger with the pumice sand than with the quartz sand.
- d) The ultimate strength of the two materials is not very different, but the strain to reach this strength is much greater with the pumice sand than the quartz sand. At the low confining stress, the pumice sand shows a slightly higher ultimate strength than the quartz sand, while at the high confining stress, the reverse is true. However, when the results of all tests are considered, the pumice sand shows a slightly higher strength than the quartz sand. The peak values from all the triaxial tests are plotted in Figure 14. These show a typical difference of about 5 degrees in the ϕ' values for the loose and dense states of the quartz sand, but very little difference in the case of the pumice sand. The ultimate strength of the pumice sand for both states approaches that of the quartz sand in the dense state. The fact that the ϕ' value for the pumice sand does not alter

significantly between loose and dense states is perhaps not surprising in view of the fact that the deviator Quartz (dense) ‘‘ (loose) Pumice (dense) ‘‘ (loose) Strain (%)

300

200

100

D

e v

i a

t o

r s

t r e

s s

(k

P a

) Confining pressure = 50 kPa 0 10 20 30 V o l u m e c h a n g e (%) 7 5 0 -3 0 10 20 30

Figure 12. Typical behaviour in triaxial tests at a low confining pressure. Quartz (dense) ‘‘ (loose) Pumice (dense) ‘‘ (loose) Strain (%)

1500

1000

500

D

e v

i a

t o

r s

t r e

s s

(k

P a

) Confining pressure = 300 kPa 0 10 20 30 10 0 -10 V o
l u m e c h a n g e (%) 0 10 20 30

Figure 13. Typical behaviour in triaxial tests at a high
confining stress. 400 800

1200

800

400 / / σ_1 σ_3 2

σ

1

σ

3

/

/ 2 QUARTZ PUMICE Dense Loose Dense Loose $\varphi = 41^\circ$ /
 $\varphi = 36.5^\circ$ / $\varphi = 41.5^\circ$ / $\sigma_1 + \sigma_3$ / / 2 $\sigma_1 + \sigma_3$
/ / 2 0 400 800 1200 1600 0 400 800 1200 1600

Figure 14. Failure envelopes from the triaxial tests. 5
0 -5 -10 -15 -20 Confining pressure (kPa)

V o

l u

m

e

c h

a n

g e

a

t f

a i

l u

r e

(%

) P u m i c e (l o o s e) Q u a r t z (d e n s e) P u m
i c e (d e n s e) Q u a r t z (l o o s e) 0 200 400 600
50 40 30 20 10 0 P u m i c e (l o o s e) Q u a r t z (d
e n s e) P u m i c e (d e n s e) Q u a r t z (l o o s e
) 0 200 400 600

Figure 15. Volume change and strain to failure in triaxial tests. Table 2. Summary of penetrometer test programme. Sand Relative density (RD) Vertical stress (kPa) Pumice Dense 50, 100, and 200 Loose Quartz Dense 50, 100, and 200 Loose Mixtures: 10% pumice, 90% quartz Dense 50, 100, and 200 25% pumice, 75% quartz Dense 50, 100, and 200

stress curves do not show peaks, at least at the higher stress levels. However, it does seem surprising that

the ϕ' value of the pumice sand is as high as that of the quartz sand in the dense state. As already mentioned in relation to Figures 12 and 14, the strain to failure and the accompanying

volume change are much greater in the case of the pumice sand than the quartz sand. To further illustrate

this difference in behaviour Figure 15 has been prepared. This summarises the data from all of the tests, and illustrates clearly the much greater strain to failure

and volume change with the pumice sand. With the pumice sand, the strain to failure in the tests at high

stress levels is somewhat arbitrary, since the deviator stress was still tending to increase even at the very

large strain values to which the tests were taken (greater than 30%).

6.4 Penetrometer tests in calibration chamber

6.4.1 Scope of testing programme

The test programme envisaged tests at three vertical stresses, namely 50, 100 and 200 kPa. At each stress

level, tests were planned on both sands in their loosest and densest states, making a total of six tests.

Some difficulties were encountered in placing the sand at its loosest and densest states; this resulted in

several extra tests being carried out. After the completion of these tests, six additional tests were carried

out on mixtures of the two sands. These were restricted to the dense state. The planned test programme

is summarised in Table 2.

6.4.2 Description of the chamber

The calibration chamber used for the tests is the original chamber developed in Australia in 1969 (Holden,

1971). This was the first flexible walled chamber designed such that boundary stresses and strains could

be measured and controlled. Figure 16 shows a schematic representation of the chamber. Similar, but

larger chambers, have been used in various parts of the world (eg Belloti et al, 1982, Jamiolkowski et al,

Figure 16. Schematic view of the calibration chamber.

1988). The chamber consists of a double-walled barrel, a base piston, and a top lid. The sand sample

is enclosed at the side and base by rubber membranes, with the side membrane sealed around the base

piston, and a platen at the top of the specimen.

Vertical stress is applied to the sample by the base piston using pressurised water. The vertical thrust

is transferred to the lid of the chamber through the top platen and resisted by an external load frame.

Radial stress is controlled by the water-filled annular space between the membrane and chamber wall.

A condition of constant radial stress can be maintained by connecting the annular space to a regulated

pressure supply, while a condition of zero lateral strain ($K = 0$ condition) can be achieved by preventing

water from entering or leaving the annular space. For the latter condition, the pressure generated in

the annular space during consolidation is continually measured and the same pressure is applied to the

water-filled space within the double-walled barrel. This ensures that the inner wall of the chamber does

not deflect under the action of the lateral stress and thus maintains an average $K = 0$ condition for the

sample. Bellotti et al (1982) present a more detailed description of a calibration chamber of this type.

A hole in the centre of the top lid permits the insertion of the penetrometer into the sand sample. The

penetrometer used was the standard device with a cross sectional area of 10 cm^2 . It was pushed into the

sample using a servo-controlled hydraulic system and a personal computer, the latter also simultaneously

recording the output during testing.

6.4.3 Size and boundary effects from the chamber

This chamber is somewhat smaller than most modern chambers. The chamber diameter is 760 mm, which

means that the ratio of chamber diameter to cone diameter (RD) is only 21. Ratios of between 35 and

50 are generally considered desirable to avoid boundary effects, especially for tests on quartz sand in a

dense state. Tests show that the smaller the RD value, (ie the smaller the chamber diameter) the lower

the value of the cone resistance. This suggests that it is not the restraining effect of the sample boundary

that influences the cone resistance; if this were the case the trend would be in the opposite direction.

Rather it is an effect that arises from the influence the chamber diameter has on the stress state. This

effect has been investigated in detail and a simple way of correcting for chamber size and the associated

stress state has been developed and applied to the test results described here. The analysis and correction

method, are described in detail by Wesley (2002).

6.4.4 Sample formation and test procedure

Previous investigations have generally adopted pluviation through air as the sample preparation tech

nique, and this procedure was initially adopted here. The pluviation technique is intended to allow the Table 3. Results of Calibration Chamber tests on pumice and quartz samples. Vertical stress σ_v (kPa) Dry Cone Type of Density D_r σ_h resistance sand (kg/m³) (%) Applied Corrected (kPa) K_o (kPa) Pumice 621 1.0 50 42.5 28 0.56 3390 " 629 9.4 100 90.4 51 0.51 4350 " 634 14.5 100 89.2 49 0.49 4900 " 624 4.2 200 186.6 76 0.38 6064 " 653 33.3 200 184.6 70 0.35 6990 " 670 49.1 100 89.5 44 0.44 4750 " 696 71.9 50 41.2 25 0.50 4000 " 697 72.8 100 88.2 40 0.40 5370 " 720 91.5 200 185.8 72 0.36 6440 Quartz 1328 4.6 50 47.2 28 0.56 1272 " 1340 11.3 100 94.4 49 0.49 2550 " 1332 6.8 200 188.8 76 0.38 5100 " 1517 98.7 50 32.8 24 0.48 7800 " 1513 96.9 100 74.2 37 0.37 11700 " 1512 96.5 200 163.7 62 0.31 16000

sand grains to fall and come to rest in a consistent and repeatable fashion. The sand is contained in

a hopper and is released through a perforated plate onto diffuser sieves. The diffuser sieves slow the

rate of fall and disperse the sand jets to produce a uniform rain of sand, forming the test sample. The

diffuser sieves are raised as the sand is deposited,

maintaining a constant raining height. Brandon and

Clough (1991) showed that the factor controlling the density of the deposited sand was the diameter

and number of holes in the perforated plate. For dense samples a small number of holes was required

and vice versa. Plates were therefore prepared for the present investigation with hole sizes and spacing

expected to produce dense and loose samples. Unfortunately, the equipment did not function as intended; the technique made possible the formation

of loose samples, but failed to produce dense samples. Various alternative techniques were investigated

to form dense specimens; the one finally adopted involved light manual tamping of the sand in thin

layers. The chamber to tamper diameter ratio was 2.6 and the sand layers were approximately 30 mm in

thickness. This method produced relative densities consistently in excess of 80%. All tests were on dry samples, "normally consolidated" under conditions of zero (K_0) lateral strain.

Each specimen was typically left for one hour to "consolidate" (creep) prior to performing the cone test.

The penetration rate was 20 mm per second. After testing, the chamber was disassembled and the sand

removed using manual methods and/or vacuum extraction. The sand immediately surrounding the cone

was carefully removed to investigation particle crushing during the tests.

6.4.5 Results of tests on pure pumice and quartz samples

The results of these tests are summarised in Table 3 and in Figures 17 to 20. Table 3 gives the initial unit

weights, relative densities, and average values of cone resistance. Figures 17 and 18 show typical results for the pumice and quartz sand respectively; these are for the

loose and dense states at a vertical confining stress of 200 kPa. In Figure 19 the cone resistance curves

from Figures 17 and 18 are shown on the same graph so that a direct comparison can be made between

the behaviour of the two sands. Figure 20 summarises all the test results in the form of the familiar graph of q_c versus vertical stress.

The vertical stress has been corrected as described earlier. There is a surprising and dramatic difference in behaviour between the two sands, as is readily

apparent in these figures. The quartz sand behaves as expected, showing large differences in cone

resistance between the loose and dense states, and steadily increasing values with confining stress. The

pumice sand, on the other hand, shows quite startling behaviour, with the following rather extraordinary

characteristics:

1. There is very little change in cone resistance between its loose and dense states, and the increase in resistance with confining stress is less pronounced than with the quartz sand.
2. The penetration resistance of the pumice sand is a little higher than that of the quartz sand in the loose state.

Figure 17. Typical calibration chamber penetrometer test on pumice sand: dense and loose samples at confining

stress of 200 kPa.

Figure 18. Typical calibration chamber penetrometer test on quartz sand: dense and loose samples at confining

stress of 200 kPa.

3. Despite showing very little difference in cone resistance, the pumice sand still shows a large difference in sleeve friction, comparable to that of the quartz sand.

This dramatic difference in behaviour can only be

attributed to the different particle strength of the two

sands. The initial interpretation put on the results was that with the quartz sand failure occurred by shear

displacement, while with the pumice sand failure occurred predominantly as a result of particle crushing.

To investigate the extent of particle crushing, samples were taken from the immediate vicinity of the

cone when emptying the chamber for the purpose of making particle size measurements. "Immediate 0.2 0.4 0.6 Cone Resistance (Mpa) D e p t h (m) Quartz, dense Quartz, loose Pumice, loose Pumice, dense 0.8 0 5 10 15 25 30

Figure 19. Cone resistance values from both sands at 200 kPa plotted on one graph. Pumice, dense Pumice, loose Quartz, dense Quartz, loose Cone resistance (MPa) 50 100 150 200 E f f e c t i v e v e r t i c a l s t r e s s (k P a) 0 5 10 15 20

Figure 20. Results of all tests showing cone resistance versus vertical effective stress.

vicinity" in this context means a zone extending not more than 1 to 2 cm from the edge of the cone. The

results of the particle size measurements are illustrated in Figure 21. A surprising amount of crushing occurs with both sands, especially in the dense state; it is only

marginally greater with the pumice than the quartz sand. This comparison is primarily qualitative, as it

was not possible to be sure that the size and location of the samples was the same in each test. Perhaps the explanation for the insensitivity of cone resistance to density with the pumice sand lies in

the shear strength behaviour revealed by the drained triaxial tests (Figures 12 and 13). Both in the dense

Figure 21. Particle crushing in penetrometer tests.

and loose state at high confining stresses, the deviator stress slowly climbs towards a peak value, but

there is no post peak decline in strength. This peak value

is essentially the same regardless of whether

the sand is initially in the dense or loose state. Thus the resistance measured in the cone test appears to

simply reflect this common "ultimate" state.

A comparison of the results from the two sands tested here with the results of similar tests on other

sands is made in Figures 22 and 23. Figure 22 shows the results plotted alongside the curves of Robertson

and Campanella (1983) for sands of varying compressibility.
3 2 1 3 2 1 0.1 0.2 0.3 0.4 0.5 V e r t i c a l e f f e c t
i v e s t r e s s (M p a) Cone resistance q_c (Mpa)
Pumice, dense Pumice, loose Quartz, dense Quartz, loose
Schmertmann (1976) Hilton Mines sand - high compressibility
Baldi et al (1982) Tucino sand - moderate compressibility
Villet & Mitchell (1981) Monterey sand - low
compressibility $D_r = 40\%$ $D_r = 80\%$ 1 2 3 0 10 20 30 40
50

Figure 22. Comparison of results with other sands of varying compressibility (based on Robertson and Campanella,

1983). 100 80 60 40 20 10
100 1000 3000 R
e l a t i v e d e n s i t y $D_r (\%)$ $q_c / (\sigma_{vo}) 0.5$
Pumice sand Quartz sand Low compressibility High
compressibility L i m i t s i n d i c a t e d b y J a m i o
l k o w s k i e t a l (1 9 8 5)

Figure 23. Comparison of results with other sands (based on Jamolkowski et al, 1985). The results appear to be reasonably compatible with the data in these figures. However, while the

pumice is clearly of high compressibility, laboratory measurements show that in its loose state its

compressibility is about four times greater than in its dense state (Figure 4). Thus if the trends indicated

in Figure 22 applied to all sands the curves for the pumice sand in its loose and dense states would be

expected to occupy significantly different positions on the chart. Figure 23 shows the data plotted on the chart of Jamolkowski et al (1985). As expected, the value

of $q_c / (\sigma_{vo})^{0.5}$ for the pumice sand does not vary with relative density, so that the trend exhibited in this

figure does not apply at all to the pumice sand. The quartz sand results conform to the trend when the

sand is in its dense state but not when it is in the loose state. The reason for this is not known for certain, M a x

i m u m M i n i m u m 0 20 40

60 80 100 1600 1200 800 400 Pumice

content (% by wt.) D r y d e n s i t y (k g / m ³)

Figure 24. Dry density of the pumice and quartz sand mixtures.

Table 4. Results of CC test on mixed pumice and quartz samples. Vertical stress σ_v (kPa) Dry Cone

Type of % density D r σ_h resistance

sand pumice (kg/m ³) (%) Applied Corrected (kPa) K o (kPa)

Mixed 10 1388 99 50 33.7 24 0.48 7200

" " 1380 96 100 76.4 38 0.38 10400

" " 1385 98 200 166.0 68 0.34 15000

Mixed 25 1225 94 50 37.1 25 0.50 5700

" " 1238 99 100 80.7 37 0.37 8500

" " 1236 98 200 173.9 68 0.34 11500

but is probably because the data on which Figure 23 is based is generally representative of harder grained

sands than the quartz sand tested here.

6.4.6 Tests on mixtures of pumice and quartz sand

These tests involved mixtures of the two sands, and were carried out to investigate the way in which

varying pumice content influences the results. Two mixtures were tested, one with 10% pumice and 90%

quartz, and the other with 30% pumice and 70% quartz. These proportions were by weight. The tests

were only done on dense samples, since it had already been established that with loose samples the cone

resistance values were very similar for both sands, and could not be expected to show much sensitivity

to the proportions of each material.

Prior to preparing the samples a series of maximum and minimum density tests was done on mixtures

of the two sands, over the full range of possible proportions. The results are shown in Figure 24. The

graph of maximum density was then used as a basis for controlling the density of the chamber samples.

The results of the chamber tests are summarised in Table 4 and Figs. 25 and 26. Figure 25 shows the

cone resistance graphs for the three stress levels, and Figure 26 shows the results in a conventional graph

relating cone resistance to vertical stress and relative density. As expected, there is a steady decrease in

cone resistance as the pumice content increases.

When the pumice content reaches 25% the cone resistance has fallen to a value about midway between

the values for 100% quartz and 100% pumice. The tests clearly show that a relatively small proportion

of pumice present in a hard-grained sand can be expected to have a significant influence on the cone

resistance.

6.4.7 Discussion

The behaviour of the pumice sand during penetrometer testing is very surprising, and means that the

main aim of this research, namely to provide correlations between cone resistance and relative density

Figure 25. Chamber test results for sand mixtures. 0
5 10 15 20 Cone

resistance (Mpa) 50 100 150 200 Effective vertical
 ical stress (kPa) 100% pumice 100% q
 uartz 10% pumice 25% pumice

Figure 26. Cone resistance versus vertical stress for sand mixtures.

of the pumice sand, was not achieved. The results of the tests are thus not good news for geotechnical

engineers wishing to use cone penetrometer tests to evaluate the properties of pumice sands (particularly

their relative density). This does not mean that the research was not useful; it is considered very useful in

warning geotechnical engineers of the pitfalls involved in the interpretation of cone penetrometer tests

in any soft grained sand.

1.2

0.8

0.4

0 0 10 20 30 Effective vertical stress /p σ
 v o / 0.1 1 10 100
 1000 Effective vertical stress /p σ v o / 0
 50 100 150 200 (a) log plot
 (b) linear plot Quiou carbonate
 sand

Figure 27. Compressibility of Quiou carbonate sand (from Pestana and Whittle, 1995).

The question arises as to what alternatives may exist for evaluating the relative density of soft-grained

sands. The laboratory tests provide an indication of a possible alternative. While there is no significant

difference in the peak (or "ultimate") strength of the pumice sand in its loose and dense states, there is

a very clear difference in its compressibility, at least at low to moderate stress levels. Thus some form

of testing that is a measure of compressibility rather than

ultimate strength would be expected to be

dependent on relative density, and be amenable to correlations with relative density. The authors were

hopeful that shear wave velocity would prove a useful measure, but tests to date have proved inconclusive.

Further research is needed.

7 COMPARISON WITH BEHAVIOUR OF OTHER CRUSHABLE SANDS

As far as the author is aware, there is no comparable data in the literature on penetrometer testing in

soft-grained sands, so that comparisons on this issue are not possible. However, there is a reasonable

amount of information available on the behaviour of "crushable" sands in laboratory compression tests,

and this can be compared with the behaviour of the pumice sand described in this paper. The most useful

collection of data is found in the Proceedings of an International Workshop on Soil Crushability held in

Ube, Yamaguchi, Japan, in 1999.

That workshop brought together data on the behaviour of sands when stresses are raised to levels at

which crushing occurs, or could be expected to occur. Data is presented for some soft grained sands, in

particular carbonate sands, but much of the data is from hard grained (silica) sands subjected to very high

pressure tests. A number of papers identify "yield" pressures or "yield" zones, and imply or comment

on a possible relationship between the onset of particle crushing and the "yield" pressure. Hyde and

Nakata (1999), for example, state that "yielding under one dimensional compression is accompanied

by particle crushing and an examination of the relation between yielding and single particle breakage is

of primary importance in increasing the understanding of the fundamental engineering characteristics

of soils”.

Most of the compression data in the Workshop is presented in the traditional form of void ratio versus

log pressure graphs. These graphs have been used to identify yield pressures. There is a danger in doing

this, as the log plot gives a distorted picture of the compressibility behaviour, which is often interpreted

as indicating a “yield” pressure when no such pressure exists. So called “yield” pressures from log plots

are often simply a function of the way the data is presented, rather than a soil property.

It has already been pointed out earlier with reference to the behaviour shown in Figures 6 to 10, that

for the pumice sand tested here there is no evidence of any connection between particle crushing and

a yield stress during vertical compression. Typical data from the Ube Workshop is re-examined here to

further consider this issue.

Figure 27 illustrates data for Quiou carbonate sand (a weak grained sand) originally presented by

Pestana and Whittle (1995). Figure 27(a) shows the data as originally presented, and Figure 27(b) shows

Figure 28. Data from Golightly (1990) on log and linear plots.

it re-plotted using a linear scale for pressure. It is very clear from the linear plot that the soil does not

show any evidence of yield stresses. Figure 28 shows data for silica sand and carbonate sand presented by Golightly (1990). It is presented

in the form originally given and also re-plotted using a linear scale. McDowell and Bolton (1998) suggest

that region 2 on the dense silica sand graph is a “well defined yielding region”. However, when viewed

on the linear plot, the question of yield zones becomes more problematical. There is clearly no yield

with the dense carbonate sand and only a faint suggestion of such a zone (around a stress of 20 MPa)

with the dense silica sample. With the loose silica sand there is also a faint indication of a yield pressure

at a little below 20 MPa. The author’s data in Figures 6 to 10 for pumice sand, together with the data in

Figures 26 and 27 for carbonate sand, suggests that soft-grained sands are unlikely to show any evidence

of yield pressures. With hard grained sands the picture is different, as the following figures will show. Figure 29 shows

compression curves for silica sands of different particle size plotted using a log scale for pressure, as

presented by Hyde and Nakata (1999), and re-plotted using a linear scale. In this case, both the log and

linear plots show evidence of yield pressures, ranging from about 10 MPa for the coarse sample to about

30 MPa for the finer sample. Figure 30 shows some more very informative data from Nakata (1999), who conducted high pressure

compression tests on four silica sands of varying particle size distribution. The particle size curves and

the test results using log and linear plots are shown in Figure 30 (a), (b) and (c) respectively. The particle

sizes of the four sands are as follows: ϕ 0.6–0.7 mm Uniformity coefficient = 1.09 ϕ 0.25–1.7 mm Uniformity coefficient = 2.63 ϕ 0.032–2.0 mm Uniformity coefficient = 3.29 ϕ 0–2.0 mm Uniformity coefficient = 9.91 They range from a very uniform material to a well-graded material. Examination of the compression

curves (with the linear pressure scale) shows a clear yield zone for the uniformly graded sample, but

as the uniformity coefficient increases from 1 to nearly 10, the existence of a yield stress declines. The

well-graded material, with the uniformity coefficient of 9.9, shows no evidence of a yield stress, and

undergoes “strain hardening” behaviour from the start of loading.

Figure 29. Compressibility of silica sand (Hyde and Nakata, 1999).

Figure 30. Behaviour of silica sand over a wide range of gradings (Nakata, 1999).

Figure 31 shows results of tests on glass bellotini and uniform silica sand (Hyde & Nakata, 1999).

The glass bellotini shows a sudden yield or “collapse” pressure, much more pronounced than that from

any tests on sand, including the uniform silica sand shown in Figure 30.

This yield phenomenon occurs very suddenly with the onset of brittle fracture of the bellotini grains.

Unlike sand particles, which are of irregular shape with random asperities, the bellotini grains are uniform

Figure 31. Behaviour of glass bellotini (Hyde and Nakata, 1999).

and spherical, so that there is no gradual crushing and fracturing process as the stress is increased. The

bellotini grains remain intact until their failure load is reached, at which point they shatter or disintegrate

into numerous fragments. Fracture of a single grain presumably transfers stress to adjacent particles,

which in turn shatter in a minor chain reaction. The main points which this analysis shows are the following:

- 1) To determine whether a yield zone or yield pressure is present in a sand (or indeed in any soil) subjected to one dimensional compression, it is necessary to plot the results using linear scales for pressure. Log plots by

themselves can easily suggest the presence of "yield" pressures when no such pressures exist.

2) Most sands do not show yield pressures during one-dimensional compression. In particular, softgrained sands such as calcareous sand and pumice sand, do not normally show yield pressures, although the more uniformly graded and hard grained varieties may sometimes show poorly defined yield pressures. The sand with the softest grains for which data are available is probably the pumice sand described in this paper; this sand shows no evidence of a yield pressure, even when tests are done on fractions of uniform particle size, either coarse or fine. Such sands undergo particle crushing from low stress levels, and in the stress levels at which crushing is most pronounced the sands show a steadily decreasing compressibility.

3) Hard grained uniformly graded sands, especially those consisting entirely of silica grains, are likely to show yield pressures; the more uniform and rounded the particles the clearer is likely to be the yield pressure. Such yield pressures are directly related to the onset of particle crushing.

4) The yield pressures evident in uniformly graded, hard grained sands are very high, generally in excess of 20 MPa, and therefore of little relevance to most engineering situations. Foundation pressures are unlikely to approach these values. Cone penetrometer tests and driven piles are probably the only common situations that could result in stresses capable of causing particle crushing in hard grained sands.

8 CONCLUSION

The principal findings from this study of a pumice sand are the following:

a) The cone resistance from penetrometer tests on the pumice sand in a calibration chamber is shown to be very similar in the loose and dense states. Cone penetrometer tests are therefore not useful as indicators of relative density of the sand.

b) Triaxial tests show that at high confining stresses, the pumice sand does not show a clear peak strength during the test. Both loose and dense samples reach the same strength, although the strain required to mobilise this "ultimate" strength is large. This behaviour is believed to account for the similar cone resistance from loose and dense

samples.

c) The pumice sand is about four times more compressible than hard grained quartz sand, in both its dense and loose state. This high compressibility is believed to be due to crushing of the soft grains.

This crushing is believed to occur at very low stress levels, although in some cases it cannot be reliably

measured until the pressure exceeds 500 or 1000 kPa.

d) Despite intense particle crushing that occurs during compression, the compression curves do not

show any evidence of yield pressures. It appears that the samples begin to "stiffen up" once particle

crushing becomes very noticeable.

e) The behaviour of pumice sand appears similar to that of other soft grained sands. Carbonate sand

also undergoes particle crushing at low stress levels, and does not show any evidence of clear yield

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Characterisation and Engineering Properties of Natural
Soils - Tan, Phoon, Hight & Leroueil (eds) © 2007 Taylor &
Francis Group, London, ISBN 978-0-415-42691-6

Small-strain stiffness of granitic and volcanic saprolites
in

Hong Kong

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ABSTRACT: While the study of the small-strain behaviour of
sedimentary soils and sands has wit

nessed significant progress, the study of the small-strain
characteristics of geomaterials derived from

the decomposition of rocks has attracted relatively less
attention. In this paper, various aspects of the

small-strain stiffness of three saprolitic soils in Hong
Kong are studied. The results of field tests (geo

physical tests and self-boring pressuremeter tests) and
laboratory tests (triaxial tests with multidirectional

shear wave velocity measurements, local strain measurements
and suction control) are presented and

discussed. Stiffness anisotropy is studied and the effects
of suction, cyclic loading, creep and sample

disturbance on small-strain stiffness are discussed. Some
unique features and characteristics of saprolitic

soils at small strains are highlighted.

1 INTRODUCTION

It is generally believed that the behaviour of saprolitic
soils (soils that preserve the original structures

inherited from the parent rock (GCO 1988)) differs considerably from that of transported soils under

similar conditions due to their special mineralogical and microstructural characteristics developed during

weathering processes. Prior to the establishment of the Geotechnical Control Office (GCO) of the Hong

Kong Government in 1977, some studies on the shear strength of granitic saprolites have been reported by

Lumb (1962, 1965). Since the late 1970s, the GCO (now Geotechnical Engineering Office (GEO)), has

carried out significant amount of research by measuring the shear strength of saturated and unsaturated

granitic saprolites through triaxial compression and direct shear tests in the laboratory and in the field

(Massey 1983, Brand et al. 1983, Shen 1985). Based on a large database of measured results from

triaxial tests, Pun & Ho (1996) reported the shear strength of intact granitic saprolites and provided a

set of generalized shear strength parameters of completely decomposed granite (CDG). Commissioned

by the GEO, Gan & Fredlund (1996) also investigated the shear strength of saturated and unsaturated

CDG using direct shear box and triaxial apparatus. The effects of matric suction on shear strength were

quantified using a simplified extended Mohr-Coulomb failure criterion.

The mechanical behaviour of loosely compacted CDG and completely decomposed volcanic (CDV)

soils has been investigated extensively in an attempt to improve the analysis and design of the use of

soil nails in loose fill slopes (Ng & Chiu 2001, 2003, Ng et al. 2004a). The effects of stress state, stress

path and soil suction on the stress-strain relationship, shear strength, and volumetric behaviour of the

materials were assessed through a series of computer-controlled triaxial stress path tests. Specific stress

path was carried out to mimic rainfall infiltration into an initially unsaturated soil slope by reducing

soil suction but keeping other stress components constant. All test results were interpreted and modelled

under a state-dependent extended critical state framework (Chiu & Ng 2003).

With the advancement in field measurement techniques (Baldi et al. 1988, Clarke 1995) and the

improvement in the accuracy of strain measurements in the laboratory (Clayton & Khattrush 1986,

Tatsuoka 1988), our understanding of the small-strain stiffness of soils has been improving continuously

and has been utilized in some routine geotechnical engineering analyses and designs over the last 15 years

(Burland 1989, Ng & Lings 1995, Atkinson 2000). Various studies revealed that the stiffness of soils

at very small strains ($\epsilon < 0.001\%$) is influenced by a number of factors including confining stress,

void ratio and soil structure. The stress-strain behaviour of natural soils is nonlinear, stress-path and

stress-history dependent at small strains ($0.001\% < \epsilon < 1\%$). Soils are generally anisotropic and behave

differently under saturated and unsaturated conditions. Despite the significant progress in understanding

the small-strain stiffness of many geomaterials, such as sedimentary soils, soft rocks, and clean sands,

relatively little attention has been paid to the small-strain behaviour of geomaterials resulting from the

decomposition of rocks. In this paper, various aspects of the small-strain stiffness of three types of saprolitic soils in Hong Kong

are considered. These soils include completely decomposed granite (CDG), completely decomposed

tuff (CDT) and completely decomposed rhyolite (CDR). The geological setting of Hong Kong is briefly

introduced in Section 2. The classification of decomposed materials and the common methods of soil

sampling in Hong Kong are presented in Sections 3 and 4, respectively. Section 5 describes the ground

conditions of the test sites and the physical properties of the tested materials. Section 6 reports the findings

of field measurements on the small-strain stiffness of decomposed granite. In Section 7, developments in

laboratory testing techniques are described. Section 7 also reports some recent findings on the stiffness

of saprolitic soils at very small to small strain levels and stiffness anisotropy at very small strains.

The effects of suction, cyclic loading, creep and sample disturbance on the small-strain stiffness of the

saprolitic soils are assessed and discussed. Finally, conclusions are drawn in Section 8. In order to focus

on soil stiffness only, further discussion on shear strength of saprolites is not included in this paper.

2 GRANITIC AND VOLCANIC ROCKS IN HONG KONG

Figure 1 shows a geological map of Hong Kong that illustrates the distributions of rock types in Hong

Kong. The predominant rock types in Hong Kong are Mesozoic volcanic rock and granite, which

constitute about 85% of rock outcrop on the land (Sewell et al. 2000). Most of the igneous rocks were

formed during the Late Jurassic and Early Cretaceous periods. Volcanic rocks occupy about 50% of

Hong Kong's surface area, forming most of the mountainous ground in Hong Kong. They include tuffs,

tuffites, lavas and sedimentary rocks. Intrusive igneous rocks in Hong Kong account for about 35% of

Hong Kong's surface area, which comprises mainly granite and granodiorite. Detailed classification and

descriptions of the volcanic and granitic rocks in Hong Kong were provided by Sewell et al. (2000). Under the influence of the warm and humid subtropical climate in Hong Kong, rock weathering

is dominated by chemical alteration processes (Irfan 1996). Weathering leads to physical and chemical Quaternary Deposits 0 4 62 8 10 12 km Hong Kong Island Chek Lap Kok Yuen Long Sai Kung Tai Po Sheung Shui Kowloon Guangdong Sheng Legend Cretaceous - Tertiary Sedimentary Rocks Jurassic / Cretaceous Granitoids Devonian - Permian Sedimentary Rocks 22°X 30'N 22°X 10'N 22°X 20'N 22°X 30'N (Shenzhen Special Economic Zone) Tsuen Wan Sha Tin 114°X 00'E 114°X30'E 114°X 20'E 113°X 50'E 113°X 50'E Lantau Island Tuen Mun Mirs Bay Deep Bay Reclaimed Land Jurassic / Cretaceous Volcanic Rocks Yen Chow Street Luen Wo Hui Site A Kowloon Ba

Figure 1. Geological map of Hong Kong showing the location of the test sites.

changes in rock material. The process starts at the rock's surface and progresses inwards along rock joints.

Ingress of water along rock joints promotes oxidation, hydrolysis and solution of the constituent minerals.

The igneous rock, such as granite, is composed mainly of quartz, feldspar and mica. Quartz is resistant

to chemical decomposition, while feldspar and mica are transformed mainly to clay minerals during the

weathering process. As weathering proceeds, the stress release as a result of the removal of the over-lying

material accelerates the rate of exfoliation (stress release jointing) and the wetting and drying processes

in the underlying fresh rock. These processes increase the surface area of the rock on which chemical

weathering proceeds, which leads to weathering profiles of

up to tens of metres thick (Irfan 1996).

Weathered rock profiles are generally overlaid with Quaternary superficial deposits of varying thickness.

Weathering effects on the mineralogical and textural aspects of weathered granites have been carried

out by Iran (1996, 1999) and Shaw (1997). Weathering mechanisms of volcanic and granitic rocks in

Hong Kong have been studied by chemical analysis, optical microscopy on thin sections and magnetic

susceptibility measurements (Ng et al. 2001). The mobility of elements of calcium, sodium, iron,

potassium, magnesium, silicon, aluminium and constitutional water has been investigated in detail.

Chemical weathering processes in Hong Kong are dominated by decomposition of feldspars and biotite,

leaching of alkali and alkaline earth metals, and enrichment of ferric oxide under prolonged subtropical

climate conditions. Based on the micropetrographic studies, it is evident that the process of weathering

gradually reduces the contents of feldspar in both volcanic and granitic rocks. On the contrary, the

amount of clay minerals, microfractures and voids increases with the degree of weathering. As expected,

the quartz remains fairly constant throughout the weathering processes. The micropetrographic index

(I_p), which examines quantitative changes of minerals, microcracks and voids in a sample, is shown to

be suitable for granitic rock but not particularly good for volcanic rock due to its fine-grained minerals

and small pore size (Ng et al. 2001).

3 CLASSIFICATION OF DECOMPOSED MATERIALS

A six-fold material decomposition grade scheme is commonly used to classify the state of decompo

sition of igneous rocks in Hong Kong (GCO 1988). The term 'decomposed' instead of 'weathered' is

used in the material classification scheme because the dominant weathering process in Hong Kong is

chemical decomposition. The general characteristics of each material decomposition grade are sum

marised in Table 1. For the purpose of engineering design, materials of Grade I to Grade III, which

Table 1. Classification of rock material decomposition grades (GCO 1988).

Rock Material

Description Grades General Characteristics

Residual Soil VI • Original rock texture completely destroyed • Can be crumbled by hand and finger pressure into constituent grains

Completely Decomposed V • Original rock texture preserved • Can be crumbled by hand and finger pressure into constituent grains • Easily indented by the point of a geological pick • Slakes when immersed in water • Completely discoloured compared with fresh rock

Highly Decomposed IV • Can be broken by hand into smaller pieces • Makes a dull sound when struck by a geological hammer • Not easily indented by the point of a geological pick • Does not slake when immersed in water • Completely discoloured compared with fresh rock

Moderately Decomposed III • Cannot usually be broken by hand; easily broken by a geological hammer • Makes a dull or slightly ringing sound when struck by a geological hammer • Completely stained throughout

Slightly Decomposed II • Cannot be broken easily by a geological hammer • Makes a ringing sound when struck by a geological hammer • Fresh rock colours generally retained but stained near joint surfaces

Fresh I • Cannot be broken easily by a geological hammer • Makes a ringing sound when struck by a geological hammer • No visible signs of decomposition (i.e. no discolouration)

generally cannot be broken down by hand, are considered as rocks. Materials of Grade IV to VI, which

generally can be broken down by hand into their constituent grains, are considered as soils. Soils of

Grade IV and Grade V are termed saprolites. Saprolite is a zone in a weathering profile that has not

undergone marked volume changes and preserves original structures inherited from the parent rock

(Fyfe et al. 2000). A weathering profile is considered to be mainly composed of completely to highly

decomposed in situ materials (grades V and IV respectively) with some residual soil (grade VI) (GCO

1988). Descriptions of the petrological changes in each material decomposition grade of granitic rocks and

volcanic rocks are given by Irfan (1996, 1999). Based on results from chemical analyses and dry density

tests on rhyolitic tuff and granitic samples, a new theoretical weathering model for calculating a mobility

index of major elements in rock and soil has been reported (Guan et al. 2001). It is found that this new

index is useful for describing changes of rock and soil during weathering and classifying the degrees of

weathering. Based on the theoretical model, a new parameter called the volume index, which combines

chemical and physical data, is suggested. This index appears to be a good indicator for classifying the

degree of weathering in Hong Kong. Attempts have been made by Massey et al. (1989) and Irfan (1996)

to correlate relationships between the degrees of weathering and engineering properties of weathered

rocks with reasonable success. The microstructure of saprolitic soil inherited from the parent rock and the weathering process leads

to considerably different behaviour in saprolitic soils

compared with transported soils. The relict primary interparticle bonding and the secondary bonding resulting from cementation in saprolitic soils affect the engineering properties, such as strength and stiffness, of saprolitic soils (Irfan 1996, 1999). Interparticle bonding is sensitive to disturbances arising from sampling and sample preparation.

Figure 2. Retractable triple-tube core-barrel (Mazier) (GCO 1987).

4 SAMPLING

The method of sampling affects the degree of sample disturbance in soil specimens. In determining properties such as shear strength and stiffness of a natural soil, intact soil samples are preferred. Rotary sampling and block sampling are considered as suitable sampling techniques to obtain sufficiently intact samples for determining shear strength and stiffness of soils derived from in situ rock decomposition (GCO 1987). In Hong Kong, the Mazier core-barrel is commonly used for soil sampling (Fig. 2; GCO 1987). It is a triple-tube core-barrel fitted with a retractable shoe. The cutting shoe and connected inner barrel projects ahead of the bit when drilling in soil and retracts when the drilling pressure increases in harder materials. This greatly reduces the possibility of any drilling fluid coming into contact with the core at or just above the point of cutting and hence minimizes sample disturbance. Moreover, the Mazier core-barrel has an inner plastic liner that protects a cored sample during transportation to a laboratory. The core diameter is 74 mm, which is compatible with the laboratory triaxial testing apparatus. Local experience commonly regards the Mazier sampling technique

as the most suitable sampling method

available for weathered granular materials at depths.

When a soil sample with the least possible disturbance is required, the block sampling technique can

be applied to retrieve the soil sample. Block samples are obtained by cutting exposed soil in trial pits or

excavations. Specially oriented samples can be obtained by block sampling for tests such as measurement

of shear strength on specific discontinuities and anisotropic soil stiffness (Ng & Leung 2006).

The comparison of the degrees of sample disturbance associated with the Mazier and block sampling

techniques and the effects of sample disturbance on the small-strain stiffness of soils are presented in

Section 7.7

5 TEST SITES AND TESTED MATERIALS

In this paper, four sites in Hong Kong are selected for studying the small-strain stiffness of saprolites.

The locations of the test sites are shown in Figure 1. Two of the sites, Kowloon Bay and Yen Chow

Street, are located on the Kowloon Peninsula where granite predominates. The site in the northern New

Territories, Luen Wo Hui, is underlain by coarse ash crystal tuff. The remaining site (Site A), where

rhyolite was encountered, is located in the northwestern New Territories. Field and/or laboratory tests

were carried out on the decomposed materials derived from the aforementioned igneous rocks.

Figures 3a and 3b show the soil profile and the results of standard penetration tests (SPT 'N' profile)

of the sites at Kowloon Bay (Ng et al. 2000) and Yen Chow Street (Ng & Wang 2001). The granite at

both sites is classified as Kowloon Granite in the Lion Rock Suite (Sewell et al. 2000). Kowloon Granite

has been dated to 138 ± 1 million years ago and represents the final pulse of plutonic activity recorded

in Hong Kong. It forms a subcircular biotite monzogranite pluton centred on Kowloon and Hong Kong

Island. The granite is uniform in texture and composition. It contains quartz and plagioclase megacrysts

set in a matrix of quartz, alkali feldspar and plagioclase with abundant biotite. At both sites, granite at

varying degree of decomposition was identified along the depth. The degree of decomposition decreases

with depth, where completely decomposed granite (CDG; Grade V) overlies highly decomposed granite

(HDG; Grade IV) and moderately decomposed granite (MDG; Grade III). The CDG at the two sites

can be described as clayey sandy silt. The physical properties and the particle size distributions of the

material are shown in Table 2 and Figure 4, respectively.

Figure 3c shows the soil profile and the SPT 'N' profile of the site at Luen Wo Hui. The Mesozoic

volcanic rock in the area belongs to the Tai Mo Shan formation of the Upper Jurassic Tsuen Wan Volcanic

Group (Sewell et al. 2000). The formation is widespread and voluminous in the northern part of the New

Territories. The lithology (physical characteristics of rocks based on grain size, mineral content and

colour) of the formation is uniform, with pale grey to dark grey lapilli-ash or coarse ash crystal tuff.

The rock is mainly made up of quartz, plagioclase, alkali feldspar and some biotite and lithic lapilli

of pale sandstone. At the depth of sample retrieval, the original rock is completely decomposed and

discoloured to reddish brown clayey silt with some crystal fragments. The pattern of foliation, which is

a result of metamorphism of the parent rock, is visible in hand specimens of CDT. The CDT samples

can be described as clayey silt (MI). The physical properties and the particle size distributions of the

material are shown in Table 2 and Figure 4, respectively. The completely decomposed rhyolite (CDR)

retrieved from Site A can be described as a firm, moist, light grey mottled with orange brown, slightly

sandy silt/clay. 0 10 20 30 40 50 60 0 50 100 150 200 SPT
'N' D e p t h (m) Fill CDT HDT MDT (c) Alluvium 0 10 20
30 40 50 60 0 50 100 150 200 SPT 'N' D e p t h (m) YCS1A
YCS1B Fill CDG HDG MDG (b) Alluvium 0 10 20 30 40 50 60 0
50 100 150 200 SPT 'N' D e p t h (m) BP1 BH2 BH6 Fill CDG
HDG MDG (a) Alluvium

Figure 3. SPT 'N' profiles at the test sites: (a) Kowloon Bay; (b) Yen Chow Street; (c) Luen Wo Hui.

Table 2. Physical properties of the tested materials. CDG (Kowloon Bay) CDG (Yen Chow Street) CDT (Luen Wo Hui)

Classification Silty sand (SM) Silty sand (SM) Clayey silt (MI)

Liquid limit (LL) (%) / / 43

Plastic limit (PL) (%) / / 29

Plasticity index (PI) (%) / / 14

Specific gravity, G_s 2.63 2.64 2.73

Moisture content (%) 19-35 15-34 17

Dry density (kg/m^3) 1210-1580 1410-1890 1730

Figure 4. Particle size distribution of the testing materials - CDG and CDT.

6 FIELD MEASUREMENTS

6.1 Field testing techniques

Although in situ measurement of soil and weak rock stiffness using geophysical methods is not very

common in Hong Kong, several studies were carried out in Hong Kong using methods including sus

pension P-S (compression and shear wave) velocity logging (Kwong 1998, Ng et al. 2000), crosshole

(Ng & Wang 2001) and downhole (Wong et al. 1998) seismic measurements. In suspension P-S velocity

logging, P-wave and S-wave velocities are measured at 1-m intervals in a single uncased borehole using

a downhole probe containing a source and two receivers. By measuring the wave travel time between the

two receivers, which are separated by 1m, the average wave velocity in the region between the receivers

can be determined. The induced strain is believed to be in the order of 0.001% or smaller, which is

sufficiently small for the determination of the elastic moduli of soils (Imai & Tonouchi 1982). For the

measurement of shear wave velocity, shear waves are transmitted vertically with horizontal polarization

and thus the velocity of the shear wave in the vertical plane ($v_s(vh)$) is measured. In crosshole seismic

measurements, a hammer provides a source of horizontally propagating, vertically polarized shear waves.

The waves are detected by two borehole-pick, three-component geophones located at the same horizontal

level as the source in two adjacent in-line boreholes. The velocity of the horizontally propagating shear

wave with vertical polarization ($v_s(hv)$) is calculated by the measured difference in travel times of the

shear wave from the source to the two geophones and the measured path length of wave propagation

along the two boreholes with geophones.

With the measured shear wave velocity ($v_s(vh)$ or $v_s(hv)$), the shear modulus in the vertical plane (G_{vh}

or G_{hv}) at very small strains can be calculated by the following equation:

where ρ is the bulk density of the soil.

In situ measurement of the variation in the shear modulus with the shear strain can be achieved

by performing self-boring pressuremeter (SBPM) tests using, for example, the Cambridge-type SBPM

(Clarke 1995). The self-boring mechanism of the pressuremeter minimizes disturbance to the surrounding

soil during installation. The Cambridge-type SBPM provides measurements of horizontal strain and

Table 3. Summary of tests performed at the test sites. Test Site Kowloon Bay Yen Chow Street Luen Wo Hui (Ng et al. (Ng & Wang) (Ng & Leung) 2000) 2001) 2006) Site A Decomposed Decomposed

Soil Type Decomposed granite tuff rhyolite

Field Test Suspension P-S N/A N/A N/A velocity logging
Crosshole seismic N/A N/A N/A measurement Self-boring N/A
N/A pressuremeter test

Laboratory Shear wave velocity N/A

Test measurement in the vertical plane using bender
elements Shear wave velocity N/A N/A N/A measurement in the
horizontal plane using bender elements Small strain
triaxial stress path test with local strain measurements

: Test was performed

N/A: No test was performed

pore pressure, thus allowing a complete stress-strain curve to be drawn. Several unload-reload loops

are generally recommended and conducted for the determination of the shear modulus of soil from the

stress-strain curve.

6.2 Shear stiffness at very small strains

Field measurements of shear wave velocity of decomposed granite along depth were taken at the Kowloon

Bay site (Ng et al. 2000) using suspension P-S velocity logging and the Yen Chow Street site using

crosshole seismic measurements (Ng & Wang 2001). A summary of the field and laboratory tests

performed at the test sites is given in Table 3. At Kowloon Bay, shear wave velocity measurements were taken across a depth from 10 m to 43 m

where alluvium and decomposed granite occupied the soil profile (Fig. 3a). At Yen Chow Street, shear

wave velocity measurements were taken on granite at various degrees of decomposition across a depth

from 29 m to 59 m. Figures 5a and 5b show the shear wave velocity profiles at Kowloon Bay and Yen

Chow Street, respectively. The shear wave velocity profiles are generally consistent with the SPT 'N'

profiles of the respective site as shown in Figures 3a and 3b. At the same depth, the shear wave velocities

measured at the two sites with similar soil profiles are consistent with each other. The shear wave velocity

of the soil profile at Kowloon Bay gradually increases with depth. In the alluvium layer, the shear wave

velocity increases from 130 m/s to 200 m/s across a depth from 10 m to 22.5 m. The shear wave velocity

of decomposed granite increases from 200 m/s at a depth of 23 m to 350 m/s at a depth of 43 m. At Yen

Chow Street, the shear wave velocity of CDG gradually increases from about 250 m/s to 350 m/s over

a depth of 29 m to 42 m. The rate of increase in the shear wave velocity increases below a depth of

42 m. Although the material at a depth between 42 m and 48 m is also classified as CDG, the degree of

decomposition is likely to decrease as the depth increases and results in a higher shear wave velocity. The

results of the shear wave velocity measurements are consistent with the results of the standard penetration

tests where a significant increase in the SPT 'N' value below a depth of 42 m is observed (Fig. 3b). The

increase in the shear wave velocity between 42 m and 48 m is a result of the increase in the soil stiffness. 0 5 10 15 20 25 30 35 40 45 50 55 60 0 100 200 300 400 500 Shear wave velocity, v_s (vh) (m/s)

D

e p

t h

(m

) 95% Confidence interval 95% Prediction interval F i l l A l l u v i u m C D G H D G (a) P-S velocity logging at BF-1 P-S velocity logging at BF-2 P-S velocity logging at BF-6 Ordinary least square 0 5 10 15 20 25 30 35 40 45 50 55 60 0 500 1000 1500 2000 Shear wave velocity, v_s (hv) (m/s) D e p t h (m) Crosshole measurement across boreholes YCS1A and YCS1B F i l l A l l u v i u m C D G H D G M D G (b)

Figure 5. Shear wave velocity profiles at: (a) Kowloon Bay; (b) Yen Chow Street.

The arrival of the refracted wave travelling in the stronger underlying stratum may also lead to an increase

in the measured shear wave velocity. The measured shear wave velocity of the limited thickness of HDG

is believed to be affected by the arrival of the refracted wave and therefore it is likely to be overestimated.

The shear wave velocity of MDG increases from 1500 m/s to 1700 m/s over a depth of 52 m to 60 m.

The shear modulus (G_{vh} or G_{hv}) at very small strains can be calculated from the measured shear wave

velocity and the bulk density of the soil using Equation 1.

The bulk density of the CDG at the two sites is about 2000 kg/m^3 . The bulk densities of HDG and MDG at Yen Chow Street are 2400 kg/m^3 and 2550 kg/m^3 , respectively. Figures 6a and 6b show the variations in shear modulus with depth at the two sites. The value of G_{vh} (or G_{hv}) increases gradually with depth across the CDG layer at both sites.

At Kowloon Bay, the value of G_{vh} increases from about 45 MPa at a depth of 23 m to about 200 MPa at a depth of 39 m. At Yen Chow Street, the value of G_{hv} of the CDG layer increases from 50 MPa to 280 MPa across a depth from 29 m to 42 m. The values of G_{hv} at depths above the strong underlying stratum (CDG/HDG boundary, HDG/MDG boundary) may be overestimated due to the arrival of the refracted wave. Along the MDG layer, the value of G_{hv} increases from 5500 MPa to 7000 MPa across a depth from 52 m to 58 m.

The shear moduli at very small strains (G_{vh} or G_{hv}) of decomposed granite at the Kowloon Bay and Yen Chow Street sites are correlated with the SPT 'N' values in Figure 7. The values of G_{hv} of granitic saprolite in Portugal (Viana da Fonseca et al. 1997) determined by crosshole seismic measurements are also shown in the figure for comparison. The value of G_{hv} of the granitic saprolite in Portugal varies from about 85 MPa to 120 MPa across a depth from 1 m to 5 m, where the SPT 'N' value increases from about 10 to 35 across the layer. A best-fit line through all the data for CDG in Hong Kong is shown in the figure. The value of G_{vh} (or G_{hv}) of CDG can be empirically correlated with the SPT 'N' value by using the following equation:

The empirical correlation between G_{vh} and the SPT 'N' value for CDG is similar to that proposed by Imai

& Tonouchi (1982) for sands ($G_{vh} = 14.4 \times N^{0.68}$). The data for CDG in Hong Kong and Portugal (Viana

da Fonseca et al. 1997) are generally consistent even though the measurements on CDG in Portugal were

taken at a shallower soil profile (Depth = 1-5 m).
 0 5 10 15 20 25 30 35 40 45 50 55 60 0 200 400 600 800 Shear modulus, G_{vh} (MPa) P-S velocity logging: BF-1 BF-2 BF-6 CDG HDG (a) Alluvium Fill 0 5 10 15 20 25 30 35 40 45 50 55 60 0 2000 4000 6000 8000 Shear modulus, G_{hv} (MPa) Depth (m) Crosshole measurement across boreholes YCS1A and YCS1B (b) Alluvium CDG HDG MDG Fill Depth (m)

Figure 6. Shear modulus profiles at: (a) Kowloon Bay; (b) Yen Chow Street. G_{vh} (or G_{hv}) = $14.3 \times N^{0.64}$ $R^2 = 0.60$ 10 100 1000 10 100 1000 SPT 'N' G_{vh} or G_{hv} (MPa) Kowloon Bay (Ng et al. 2000): BF-1 BF-2 BF-6 Yen Chow Street (Ng & Wang 2001) Portugal (Viana da Fonseca 1997)

Figure 7. G_{vh} vs. SPT 'N' for completely decomposed granite (CDG).
 0 200 400 600 800 1000 0.001 0.01 0.1 1 Shear strain, ϵ_s (%) $G_{sec} / \sqrt{p'}$ Depth = 30 - 39 m test1 test2 test3 test4 test5 test6 test7 test8 test9 test10 test11 test12 test13 test14 test15 0 200 400 600 800 1000 0.001 0.01 0.1 1 Shear strain, ϵ_s (%) $G_{sec} / \sqrt{p'}$ Depth = 39.4 m Depth = 42.4 m Depth = 45.0 m Depth = 48.5 m (a) (b) From P-S velocity logging From crosshole seismic measurements

Figure 8. Stiffness-strain relationships of decomposed granite from self-boring pressuremeter tests: (a) Kowloon

Bay; (b) Yen Chow Street.

6.3 Variations in shear stiffness with shear strain

Self-boring pressuremeter (SBPM) tests were carried out at the Kowloon Bay and Yen Chow Street

sites to study the variations in the secant shear modulus with shear strain of decomposed granite (Ng

et al. 2000, Ng & Wang 2001). SBPM tests were performed using the Cambridge-type SBPM at both

sites. Fifteen tests were conducted along a depth of 30 m to 39 m at Kowloon Bay, while four tests were

performed across a depth of 39 m to 49 m at Yen Chow Street. Figure 8a shows the relationship between

the normalized secant shear modulus ($G_{sec} / \sqrt{p'}$, where p' is the mean effective stress) and the shear strain

(ϵ_s) interpreted from the third cycle of the unload-reload loop of the tests at Kowloon Bay. The variations

in $G_{sec} / \sqrt{p'}$ with ϵ_s at Yen Chow Street are shown in Figure 8b, where the results were derived from

the last complete unloading loop. It was previously found that the secant shear modulus derived from

the last complete unloading loop is consistent with that derived from each unload-reload loop (Ng &

Wang 2001), thus it is believed that the results shown in Figures 8a and 8b are comparable. At the two

sites, the relationships between $G_{sec} / \sqrt{p'}$ and ϵ_s for decomposed granite are generally consistent. The

stiffness-strain relationship is highly non-linear. The value of $G_{sec} / \sqrt{p'}$ decreases significantly from about

300 to about 50 as ϵ_s increases from 0.02% to 1%. The range of the shear modulus at very small strains

(G_{vh} or G_{hv}) determined from seismic measurements is also shown in the figures. The measured value

of G_{vh} or G_{hv} corresponds to the maximum shear modulus of the decomposed granite.

7 LABORATORY MEASUREMENTS

7.1 Laboratory testing techniques

Shirley & Hampton (1978) developed a shear wave transducer utilizing piezoceramic bender elements

to measure the shear wave velocities and thereby the shear moduli of soils at very small strains. Bender

elements can be incorporated into various laboratory testing devices to determine the shear wave velocity

and shear modulus of soils. These devices include oedometer cells, resonant column devices, triaxial

cells and many others (Dyvik & Madshus 1985, Thomann & Hryciw 1990). In an attempt to investigate

the stiffness anisotropy of soils at very small strains, multidirectional shear wave velocity measure

ment techniques have been developed. In such techniques, bender elements are installed in different

directions in laboratory testing equipment, such as oedometer cells and triaxial testing apparatus. In

an oedometer cell, bender elements can be installed in the top cap and base pedestal to determine the

velocity of vertically propagated shear wave with horizontal polarization ($v_s(vh)$) (Thomann & Hryciw

1990, Shibuya et al. 1997). Bender elements can also be installed across the opposite faces of the ring

of an oedometer cell to determine the velocity of horizontally propagated shear waves with horizontal

polarization ($v_s(hh)$) (Jamolkowski et al. 1995). In a triaxial test, bender elements can be installed in the

top cap and base pedestal to determine the velocity of vertically propagated shear waves with horizontal

polarization ($v_s(vh)$). This technique has been widely adopted to determine the shear wave velocity of

soils in triaxial tests, except naturally decomposed geomaterials (Dyvik & Madshus 1985, Viggiani &

Atkinson 1995, Jovic'ic' & Coop 1998). Recent advances in laboratory testing techniques have enabled the

measurement of the velocity of horizontally propagated shear waves with vertical and horizontal polar

ization ($v_s(hv)$ and $v_s(hh)$) in triaxial tests

(Fioravante 2000, Kuwano et al. 2000, Pennington et al. 2001,

Ng et al. 2004b). Assessment of soil stiffness in laboratory tests requires accurate measurement of strains at low levels.

External measurements of strains in triaxial tests have several sources of errors. These include bedding

errors at the ends of the specimen, alignment errors, seating errors and system compliance (Scholey et al.

1995). Assessment of soil stiffness in a range of strains at low levels is accomplished through the use

of local strain transducers. The use of local strain transducers can eliminate the errors and improve the

accuracy and resolution of strain measurements. The types of local strain transducers include submersible

LVDT (Brown & Snaith 1974), proximity transducers (Hird & Yung 1989), inclinometer gauges (Burland

& Symes 1982), Hall effect local strain transducers (Clayton & Khatrush 1986, Clayton et al. 1989, Ng

et al. 1998) and local deformation transducers (LDT) (Tatsuoka 1988, Ng et al. 1995). Scholey et al.

(1995) and Yimsiri & Soga (2002) provided detailed reviews of these transducers. Figure 9a shows a computer-controlled triaxial stress path apparatus equipped with local strain trans

ducers and bender elements used in a study of the anisotropic small strain stiffness of a completely

decomposed tuff (CDT) at the Hong Kong University of Science and Technology (Ng et al. 2004b, Ng &

Leung 2006, Leung 2005). Local axial and radial strains at the mid-height of the specimens were meas

ured by Hall Effect transducers. A mid-plane pore pressure probe was used to measure the pore pressure

at the mid-height of the specimens. Three pairs of bender elements were used to measure the shear wave

velocity in three orthogonal planes of a soil specimen. A pair of bender elements was inserted into the

top and bottom ends of a specimen to determine the shear wave velocity of vertically transmitted shear

waves with horizontal polarization, $v_s(vh)$. Two pairs of bender elements were inserted at the mid-height

of a specimen to determine the shear wave velocity of the horizontally transmitted shear waves with

horizontal and vertical polarization, $v_s(hh)$ and $v_s(hv)$. Ng & Yung (2005) modified the triaxial apparatus shown in Figure 9a for testing unsaturated soils

using the axis translation technique (Hilf 1956). Figure 9b shows a schematic diagram of the modified

triaxial apparatus. Air pressure is controlled through a coarse, low air-entry value corundum disk placed

on the top of the soil specimen, whereas water pressure is controlled through a saturated, high air-entry

value (3 bars) ceramic disk sealed to the pedestal of the apparatus. A base pedestal with spiral-shaped

Figure 9. Triaxial apparatus with multidirectional shear wave velocity measurements and local strain measurements

for testing: (a) saturated soils; (b) unsaturated soils.

drainage ducts was designed to facilitate the removal of any diffused air through the high air-entry

value ceramic disk. A 16 mm-diameter recess in the center was designed to house a bender element for

shear wave velocity ($v_s(vh)$) measurements. A pre-drilled, high air-entry value ceramic disk was placed

in the pedestal and sealed on the inner and outer circumferences of the disk with epoxy resin. Shear

wave velocity ($v_s(hh)$ and $v_s(hv)$) measurements across the mid-height of a soil specimen and local strain

measurements using Hall Effect transducers were also

installed as shown in Figure 9a. 0 50000 100000 150000
 200000 250000 300000 350000 400000 0 50 100 150 200 250 300
 350 400 p'/p_r G_{vh}/p_r Depth = 23.0 - 24.0 m Depth =
 27.5 - 28.5 m Depth = 36.5 - 37.5 m Depth = 44.0 - 45.0 m
 Depth = 53.0 - 54.0 m Crosshole seismic measurements Depth
 = 23.0 - 24.0 m Depth = 27.5 - 28.5 m Depth = 36.5 - 37.5 m
 Depth = 44.0 - 45.0 m Depth = 53.0 - 54.0 m (Ng & Wang
 2001) Computed shear modulus ($G_{vh}/p_r = 7641e^{-0.641}$
 (p'/p_r)^{0.448}) - Measured shear modulus -

Figure 10. Normalized shear modulus at very small strains
 (G_{vh}/p_r) of CDG at Yen Chow Street. Table 4. Regression
 parameters C, a, and n for CDG and Ottawa sand. Soil type C
 a n r 2 CDG (32% silt, 18% clay) 7641 -0.641 0.448 0.90
 Ottawa sand (Salgado et al. 2000) 0% silt 547 -1.051 0.443
 0.97 5% silt 410 -1.044 0.458 0.95 10% silt 135 -2.376
 0.557 0.96 15% silt 101 -2.069 0.715 0.94

7.2 Small-strain stiffness of decomposed granite

7.2.1 Shear modulus at very small strains

The shear modulus at very small strains (G_{vh}) of CDG was
 determined from shear wave velocity measure

ments using bender elements in a triaxial apparatus (Wang &
 Ng 2005). Mazier specimens taken from the

site at Yen Chow Street at depths between 23 m and 54 m
 were tested. The specimens were isotropically

consolidated to the estimated in situ mean effective stress
 for shear wave velocity measurement. The

specimens were then subjected to 50 kPa increments of mean
 effective stress for the determination of

the shear modulus at those stress levels. Figure 10 shows
 the normalized shear modulus at very small

strains (G_{vh}/p_r) plotted against the normalized mean
 effective stress (p'/p_r), where p_r is the reference

pressure taken as 1kPa. The test data were interpreted
 using an empirical nonlinear equation suggested

by Salgado et al. (2000) as shown below:

where C, a, and n are regression constants and e is the
 void ratio. The values of the regression constants

are summarized in Table 4. The results of the regression analysis on the test data of Ottawa sand (Salgado

et al. 2000) are also shown in the table for comparison. The C value for CDG is significantly larger than that for the Ottawa sand mixtures. This seems to

imply that the intact CDG has a higher G_{vh} than has the artificial Ottawa sand mixtures for a given void

ratio (e) and mean effective stress (p'). The α value of CDG is less than that of Ottawa sand, indicating that

the void ratio (e) has less significant influence on G_{vh} in intact CDG than that in Ottawa sand mixtures

with high fine contents. This difference may be attributed to the presence of soil fabric and bonding in

the intact CDG inherited from the parent rock. One distinctive feature for saprolitic soils is the structure

and bonding resulting from weathering processes (Blight 1997). It has been shown that the shear wave

velocity, and thus the shear modulus, is less sensitive to void ratio changes in cemented soil (Yun &

Santamarina 2005).

The shear modulus determined in crosshole seismic measurements (G_{hv}) (Ng & Wang 2001) conducted

at the same depths and at the same site where the Mazier specimens were extracted for laboratory tests

are shown in Figure 10. The shear modulus determined in the laboratory tests (G_{vh}) is about 50% to

80% of that measured in the field (G_{hv}). The in situ horizontal stresses may be higher than the estimated

horizontal stresses (assuming $K_0 = 0.4$), which result in a higher stiffness determined in the field. The

discrepancies in the measured shear modulus may also be attributed to the loss of soil structure and

bonding in the laboratory specimens as a result of sample

disturbance.

7.2.2 Variations in shear modulus with shear strain

Wang & Ng (2005) studied the influence of stress paths and recent stress history on the small-strain

stiffness of CDG from a series of constant p' triaxial compression and extension tests. The recent stress

history is defined as the rotation of the current stress path from the penultimate stress path (Atkinson

et al. 1990a). In addition to the recent stress history, the stiffness of soils is influenced by the state

of the soil, for example, the stress state and the void ratio. In the study of stiffness degradation of

soils, normalization is required in order to compare the results from different tests and materials. To

account for the effects of the stress state and void ratio, the shear modulus determined in the triaxial

stress path tests presented in this paper is normalized by the shear modulus at very small strains, G_{vh} ,

obtained from the shear wave velocity measurements. This way of normalization is often adopted in

presenting the shear modulus of soils determined in monotonic and cyclic tests (Stokoe et al. 1995,

Tatsuoka 2001).

Constant p' triaxial compression tests were carried out on the Mazier specimens of CDG taken from

various depths at Yen Chow Street (Wang & Ng 2005). The tests were performed in a triaxial apparatus

equipped with local strain transducers (Hall-effect transducers). Figure 11a shows the variations in the

normalized secant shear modulus (G_{sec}/G_{vh}) with deviatoric strain (e_q) of the constant p' compression

tests. The value of G_{vh} at each mean effective stress level (p') was determined from the shear wave

velocity measurement in another series of tests using bender elements embedded in the end platens of

the triaxial apparatus. $G_{sec}/G_{vh} = 1$ thus represents the elastic shear modulus. As expected, the shear

modulus decreases as the deviatoric strain increases. The results among the five tests are scattered,

which may be the result of the sample variability of the CDG specimens along the depth from 23 m to

54 m. The average shear modulus degradation curve is shown in the figure. The average curve shows

that the shear modulus of CDG in the compression tests drops to about 30% of the initial value (G_{vh}) at

a deviatoric strain of 0.01% and further reduces to 15% of the initial value (G_{vh}) at a deviatoric strain

of 0.1%.

A series of constant p' triaxial extension tests were also conducted on Mazier specimens of CDG

taken from the same site (Wang & Ng 2005). Figure 11b shows the shear modulus degradation curves

of the constant p' extension tests. The value of G_{vh} for normalization at each mean effective stress

level (p') was the same as that in the compression tests. An average shear modulus degradation curve

is shown in the figure. It can be seen that the shear modulus of CDG along the extension path reduces

to about 50% and 20% of the initial value (G_{vh}) at deviatoric strains of 0.01% and 0.1%, respectively.

The material shows a stiffer response along the extension path, where G_{sec}/G_{vh} is about 60% higher than

that along the compression path at a deviatoric strain of 0.01%. The difference in G_{sec}/G_{vh} between the

extension and compression paths diminishes to about 20% at a deviatoric strain of 0.1% and vanishes at a

deviatoric strain of 1%. The higher shear modulus along the extension path may be attributed to stress path

reversal. During the weathering process, some of the original minerals in the granite were decomposed

and leached. This weathering process is encouraged by sufficient precipitation in Hong Kong and leads

to a significant volume loss and ground settlement (Goodman 1993). This leaching process is analogous

to applying compression loads to soil specimens in such a way that the soils are subjected to similar

downward deformation. When the intact soil specimens were loaded along the extension path, stress path

reversal took place and resulted in a higher stiffness response (Atkinson et al. 1990a). On the contrary,

the compression tests followed the continuing stress path of the natural weathering process. There was

no stress path reversal, which resulted in a lower stiffness response.

0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
0.001	0.01	0.1	1	Deviatoric strain, ϵ_q (%)						
G_{sec} / G_{vh}										
$p' = 100 \text{ kPa}$										
$p' = 150 \text{ kPa}$										
$p' = 200 \text{ kPa}$										
$p' = 250 \text{ kPa}$										
$p' = 300 \text{ kPa}$										
Average										

0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
0.001	0.01	0.1	1	Deviatoric strain, ϵ_q (%)						
G_{sec} / G_{vh}										
$p' = 100 \text{ kPa}$										
$p' = 150 \text{ kPa}$										
$p' = 200 \text{ kPa}$										
$p' = 250 \text{ kPa}$										
$p' = 300 \text{ kPa}$										
Average										

(a) (b)

Figure 11. G_{sec} / G_{vh} vs. ϵ_q of CDG: (a) Compression tests; (b) Extension tests.

7.2.3 Comparison of the shear modulus-shear strain relationship determined in triaxial tests and in-situ SBPM tests

Figure 12 compares the variations in the shear modulus of CDG with the shear strain determined in

triaxial tests and in SBPM tests. The results of the constant p' ($p' = 200 \text{ kPa}$) triaxial compression tests

on the Mazier specimens of CDG at Kowloon Bay (Depth = 36 m) (Ng et al. 2000) are included for

comparison. The upper and lower bounds of the measurements are shown. The CDG at Yen Chow

Street shows a higher shear modulus in the field tests than in the laboratory tests. The reasons for the

discrepancy include sample disturbance and differences in stress history. Some degree of damage to

the bonding and structure of natural CDG is inevitable during sampling, transportation, storage and

specimen preparation. The loss of bonding and structure in soils results in a significant reduction in

soil stiffness (Atkinson et al. 1990b, Cuccovillo & Coop 1997). The shear modulus determined in the θ 100

200

300

400

500

600

700

800 0.001 0.01 0.1 1 Shear strain, ϵ_s (%)

G

s e

c / p

' Laboratory measurements Constant p' compression (Ng et al. 2000) Constant p' compression (Ng & Wang 2001) Constant p' extension (Ng & Wang 2001) Field measurements SBPM tests (Ng et al. 2000) SBPM tests (Ng & Wang 2001)

Figure 12. Comparison of the field and laboratory measurements on the variations in the shear modulus of CDG

with shear strain.

SBPM tests is derived from the unloading stress-strain curve. Unloading involves stress path reversal and

thus results in a higher shear modulus. While the upper bounds of the shear modulus determined in the

SBPM tests conducted at Yen Chow Street and Kowloon Bay are similar, the shear modulus determined

in the laboratory tests (constant p' triaxial compression) on the CDG at Kowloon Bay is higher than that

determined in similar tests on the CDG at Yen Chow Street. It should be noted that the void ratios of

the CDG specimens from the two sites are similar at the same mean effective stress level ($p' = 200$ kPa).

The higher shear modulus determined in the laboratory tests on the CDG at Kowloon Bay might be due

to a lower degree of decomposition of the CDG sampled in Kowloon Bay.

7.2.4 Study of stiffness anisotropy of natural CDG using Mazier specimens

It is well documented that the shear wave velocity of soil depends on the stresses in the direction of wave

propagation and particle motion and is independent of the stress normal to the plane of shear (Roesler

1979). Assuming that the effects of the two stresses in the plane of shear on the shear wave velocity are

similar (Bellotti et al. 1996), a semi-empirical relationship between the shear modulus and the state of

the soil can be expressed as follows (Ng & Leung 2006):

where

i : a subscript indicating the direction of wave propagation, which is denoted by h (horizontal direction) or by v (vertical direction);

j : a subscript indicating the direction of particle motion, which is denoted by h (horizontal direction) or by v (vertical direction);

G_{ij} : the elastic shear modulus in the i - j plane;

$v_{s(ij)}$: the shear wave velocity in the i-j plane;

ρ : the bulk density of the material;

C_{ij} : a constant reflecting the soil fabric in the i-j plane, with dimension m/s;

$F(e)$: a void ratio function relating shear modulus to the void ratio; Table 5. The shear moduli, G_{hv} and G_{hh} , of CDG (Mazier specimen). p' (kPa) q (kPa) σ'_v (kPa) σ'_h (kPa) G_{hv} (MPa) G_{hh} (MPa) 80 80 133 53 82 65 160 160 267 107 126 110 400 400 667 267 275 220 10 100 1000 1000 10000 100000 1000000 $\sigma'_i \times \sigma'_j$ (kPa²) G_{ij} (MPa) G_{hh} G_{hv}

Figure 13. Variations in G_{ij} with $\sigma'_i \times \sigma'_j$ of CDG (Mazier specimen).

ρ_w : the density of water;

p_r : the reference pressure, taken as 1kPa in this study;

σ'_i : the effective principal stress in the direction of wave propagation;

σ'_j : the effective principal stress in the direction of particle motion;

n : an empirical stress exponent.

By using the equipment shown in Figure 9a, multidirectional shear wave velocity measurements were

taken on a Mazier specimen of CDG taken from Yen Chow Street at a depth of 27 m. Measurements

were taken at anisotropic stress states of $p' = q = 80$, 160 and 400 kPa. The measured values of the

shear moduli in the vertical (G_{hv}) and the horizontal planes (G_{hh}) are summarized in Table 5. Under the

influence of a higher vertical effective stress, G_{hv} is on average higher than G_{hh} by about 22% at each

of the stress states ($p' = q = 80$, 160 and 400 kPa), showing stress-induced anisotropy. Figure 13 shows

the variations in G_{hv} and G_{hh} with the product of

effective stresses in the plane of shear ($\sigma'_{i \times \sigma'_{j}}$).
By

plotting G_{hv} and G_{hh} against ($\sigma'_{i \times \sigma'_{j}}$), the effect of horizontal and vertical effective stresses on the shear

moduli can be accounted for. Best-fit lines through the data of G_{hv} and G_{hh} are shown in the figure.

The best-fit lines through the data of G_{hv} and G_{hh} are close to each other. This suggests that under

isotropic stress condition, i.e. $\sigma'_h = \sigma'_v$ and $\sigma'_h \times \sigma'_v = \sigma'_h \times \sigma'_h$, the values of G_{hv} and G_{hh} are similar. The

inherent stiffness anisotropy (degree of anisotropy under isotropic stress state) of CDG thus appears not

to be obvious from this limited set of test results. It has been demonstrated that the measured degree

of inherent stiffness anisotropy of CDT is lower in Mazier specimens than in block specimens due to

a higher degree of sample disturbance (Ng & Leung 2006). The effects of sample disturbance on the

small-strain stiffness and stiffness anisotropy of soil are further discussed in Section 7.7. As the CDG

specimen was obtained by Mazier sampling, the anisotropic structure of CDG, if any, may be damaged
 $10 \ 100 \ 1000 \ 1000$
 $10000 \ 100000 \ 1000000 \ \sigma'_{i \times \sigma'_{j}}$ (kPa²) G_{ij} (MPa)
 Block specimen Mazier specimen Isotropic Anisotropic G_{hh} G_{hv} G_{vh} G_{hh} G_{hv} G_{vh} G_{hh} G_{hv} G_{vh} G_{hh} G_{hv} G_{vh}

Figure 14. Variations in G_{ij} with $\sigma'_{i \times \sigma'_{j}}$ of CDT (block and Mazier specimens) (Ng & Leung 2006).

by the sample disturbance. Further tests are required to study the stiffness anisotropy of CDG using

specimens with the least degree of sample disturbance, i.e. block specimens.

7.3 Stiffness anisotropy of completely decomposed tuff (CDT)

7.3.1 Inherent stiffness anisotropy at very small strains

Block and Mazier samples of CDT were taken from an excavation site in Luen Wo Hui (Fig. 1) for the

investigation of stiffness anisotropy at very small strains in the laboratory (Ng & Leung 2006). The tests

were conducted in a computer-controlled triaxial apparatus equipped with bender elements and local

strain transducers as shown in Figure 9a.

Block and Mazier specimens of CDT were subjected to isotropic effective stresses in the ranges of

80 kPa to 400 kPa and 100 kPa to 400 kPa, respectively. Shear wave velocities in three orthogonal planes,

$v_s(hh)$, $v_s(hv)$ and $v_s(vh)$, were measured to determine the inherent anisotropy of the material. Figure 14

shows the shear moduli (G_{hh} , G_{hv} and G_{vh}) of the block and Mazier specimens of CDT under isotropic

stress states. The shear moduli of CDT measured under anisotropic stress states are also shown in the

figure and the results are discussed in the next section. CDT shows stiffness anisotropy, with the shear

modulus in the horizontal plane (G_{hh}) higher than the shear modulus in the vertical plane (G_{hv} or G_{vh}).

The values of G_{vh} , G_{hv} and G_{hh} and the degree of stiffness anisotropy expressed as G_{hh}/G_{hv} of the block

and Mazier specimens are summarized in Table 6. The average values of G_{hh}/G_{hv} of the block and Mazier

specimens were 1.48 and 1.36, respectively. The shear moduli, G_{hh} , G_{hv} and G_{vh} , of the block specimen

are consistently higher than those of the Mazier specimen. The Mazier specimen shows a lower degree of

anisotropy and lower shear moduli due to sample disturbance, which is further discussed in Section 7.7.

It should be noted that in an ideal elastic continuum, G_{vh} equals G_{hv} . In both specimens, G_{vh} does not

equal G_{hv} . A similar observation is made in the studies of stiffness anisotropy of sands and clays (Kuwano

et al. 1999, Pennington 1999). The discrepancy may be due to the deviation from the assumption of the

ideal continuum of soil specimens.

Table 7 summarizes some recent findings on the degree of inherent stiffness anisotropy of different

geomaterials by shear wave velocity measurements. While it has been shown that the degree of inherent

stiffness anisotropy of reconstituted or air-pluviated materials is higher in finer materials (i.e. G_{hh}/G_{hv}

Table 6. Laboratory measurements of the shear moduli, G_{vh} , G_{hv} and G_{hh} , of CDT (Ng & Leung 2006). Stress state Shear modulus Degree of

Type of anisotropy

specimen p' (kPa) q (kPa) Void ratio G_{vh} (MPa) G_{hv} (MPa) G_{hh} (MPa) G_{hh}/G_{hv} Isotropic -

Block 80 0 0.56 71 82 127 1.54 100 0 0.55 77 70 127 1.40
200 0 0.54 109 131 194 1.47 300 0 0.53 139 163 254 1.56 400
0 0.52 178 194 277 1.43

Mazier 100 0 0.76 42 55 76 1.36 200 0 0.74 62 78 107 1.38
300 0 0.72 77 108 149 1.38 400 0 0.70 92 124 165 1.32
Anisotropic -

Block 80 80 0.58 75 86 106 1.24 100 100 0.58 79 86 109 1.27
200 200 0.56 117 126 158 1.26 300 300 0.55 154 157 203 1.30
400 400 0.54 176 194 239 1.23

Mazier 100 100 0.83 42 56 65 1.16 200 200 0.80 63 77 90
1.16 300 300 0.78 84 97 119 1.22 400 400 0.76 109 130 136
1.04

is higher in clay than in silt and sand; Kuwano et al. 1999), the degree of inherent stiffness anisotropy

of the block specimen of natural CDT ($G_{hh}/G_{hv} = 1.48$) is of a similar order to some natural clays such

as London clay. The relatively high degree of anisotropy of CDT is possibly attributed to some geologic

processes such as metamorphism of the parent rock mass of CDT. The tuff in the area is slightly schistose

as a result of metamorphism and possesses a metamorphic matrix dominated by finely divided quartz and

sericite with a weak orientation (Ho & Langford 1987, Langford et al. 1989). The anisotropic fabric is

retained in the saprolite of the tuff and results in stiffness anisotropy. Shear wave velocity measurements

on saturated reconstituted CDT showed that the shear modulus at very small strains in the horizontal

plane is similar to that in the vertical plane (Yung 2004). This shows that the anisotropic soil structure

of natural CDT resulting from geologic processes is responsible for the stiffness anisotropy. A higher

degree of inherent stiffness anisotropy in natural soil is also exhibited in Pentre silt (Kuwano et al.

1999), a natural glacio-lacustrine clayey-silt possessing a laminated macro-structure, where the degree

of inherent anisotropy ($G_{hh}/G_{hv} = 1.7$) is higher than in some natural clays, such as London clay and

Pisa clay ($G_{hh}/G_{hv} = 1.4-1.5$). In air-pluviated sand and reconstituted clay, the inherent anisotropy is due

to the depositional fabric and grain characteristics of soil particles (Arthur & Menzies 1972, Kuwano

et al. 1999). The degree of inherent anisotropy of air-pluviated sand ranges from 1.1 to 1.2, while that

of reconstituted clay is in the range of 1.2 to 2.0.

7.3.2 Stress-induced stiffness anisotropy of CDT at very small strains

The degree of stiffness anisotropy in the block and Mazier specimens of CDT under anisotropic stress

states has been reported by Ng & Leung (2006). Specimens were subjected to anisotropic effective

stresses ranging from $p' = q = 80$ kPa to $p' = q = 400$ kPa. The stress ratio ($\eta = p'/q$) was kept constant at

$\eta = 1.0$, corresponding to $K = \sigma'_h / \sigma'_v = 0.4$. Shear wave velocities in three orthogonal planes, $v_s(hh)$, $v_s(hv)$

and $v_s(vh)$, were measured and the corresponding shear moduli, G_{hh} , G_{hv} and G_{vh} , were calculated ($G_{ij} = \rho$

$v_s(ij)$). Figure 14 shows the shear moduli (G_{hh} , G_{hv} and G_{vh}) of the block and Mazier specimens of CDT

under anisotropic stress states. The shear moduli determined in the anisotropic stress states are generally

consistent with the data obtained in the isotropic stress state after considering the effective principal

stresses in the shear planes. This implies that the shear wave velocities depend on the two effective

stresses in the shear planes as suggested in Equation 4. The higher value of G_{hh} in both specimens

is due to the horizontal layering structure of CDT as discussed in Section 7.3.1. The value of G_{hv} is

generally higher than G_{vh} in both specimens, but the difference decreases with increasing stress level.

The increasing vertical effective stress enhances the development of the force chains through the soil

particles in the vertical direction, resulting in a relative increase in G_{vh} (Jardine et al. 1999).

Table 7. Inherent stiffness anisotropy of different geomaterials (Ng & Leung 2006). Sample preparation

Material	method/nature	p' (kPa)	e	G_{hh} / G_{hv}	Reference
Speswhite kaolin	Reconstituted	250-600	N/A	1.7 *	Jovicic' & Coop (1998)
Speswhite kaolin	Reconstituted	300	N/A	2.0 *	Kuwano et al. (1999)
London clay	Reconstituted	400	N/A	1.24 *	Jovicic' & Coop (1998)

HPF4 silt Reconstituted 400 N/A 1.5 * Kuwano et al. (1999)

Toyoura sand Air pluviation 100 N/A 1.2 * Kuwano et al. (1999)

Ticino sand Air pluviation 25-400 0.81 1.1 * Fioravante (2000)

Ticino sand Air pluviation 100-300 0.80-0.81 1.21 ‡ Bellotti et al. (1996)

Kenya sand Air pluviation 10-700 1.3 1.2 * Fioravante (2000)

Pisa clay Natural N/A N/A 1.4 ‡ Jamiolkowski et al. (1995)

Panigaglia clay Natural N/A N/A 1.6 ‡ Jamiolkowski et al. (1995)

London clay Natural 120-400 N/A 1.5 * Jovicic' & Coop (1998)

Cambridge Gault clay Natural 90 0.81 2.01 * Pennington (1999)

Pentre silt Natural 50 N/A 1.7 * Kuwano et al. (1999)

Edogawa Pleistocene Natural N/A 0.80-0.95 1.1 § Nishio & Katsura (1995)

deposit (sand)

Completely decomposed Natural 80-400 0.52-0.58 1.48 * Ng & Leung (2006)

tuff (CDT)

* Bender elements in triaxial apparatus

‡ Geophones in calibration chamber

‡ Bender elements in oedometer

§ Bender elements mounted in bearing plates placed at opposite ends of soil block

N/A Information is not available

The measured shear moduli are summarized in Table 6. As in the tests under isotropic stresses, G_{hh}

is higher than G_{hv} and G_{vh} in both specimens under anisotropic stresses. This is not expected as it

would be intuitive to think that the shear modulus in the vertical plane would be higher than that in the

horizontal plane under higher vertical effective stress conditions (refer to Equation 4) if there were no

prior knowledge of any anisotropic soil fabric. The effect of a stronger layering structure in the horizontal

plane on the shear modulus (i.e. $G_{hh} > G_{hv}$) prevails even under a higher vertical effective stress condition

(i.e. $K = 0.4$). The average values of the degree of stiffness anisotropy (G_{hh}/G_{hv}) are 1.26 and 1.15 for

block and Mazier specimens, respectively. The reduced degree of stiffness anisotropy in the anisotropic

stress state compared with the isotropic stress state is due to the effect of a higher vertical effective stress

in the anisotropic stress state. As shown in Equation 4, a higher vertical effective stress increases the

shear modulus in the vertical plane, G_{hv} , and thus reduces the degree of stiffness anisotropy (G_{hh}/G_{hv}).

The shear moduli (G_{hh} , G_{hv} and G_{vh}) of the block specimens are again consistently higher than those of

the Mazier specimen due to the lower degree of sample disturbance of the block specimens. The effects

of sample disturbance are addressed in Section 7.7.

7.4 Effects of suction on anisotropic stiffness

Using the equipment shown in Figure 9b, Ng & Yung (2005) investigated the anisotropic stiffness of a

reconstituted CDT retrieved from Luen Wo Hui site (Fig. 1) at various matric suctions ($u_a - u_w = 0, 50,$

100 and 200 kPa), where u_a and u_w are the pore air pressure and the pore water pressure, respectively.

It should be noted that $u_a - u_w = 0$ represents the saturated case. The shear wave velocities in three

orthogonal planes, $v_s(vh)$, $v_s(hv)$ and $v_s(hh)$, were measured at different matric suctions. Details of the

testing procedures are described by Ng & Yung (2005). Figure 15 shows the variations in shear wave

velocity with matric suction ($u_a - u_w$) for different values of net mean stress ($p - u_a$), where p is the

mean total stress. The shear wave velocities increase with the two stress state variables, matric suction

and net mean stress. The shear wave velocities increase non-linearly with increasing matric suction.

Within the air-entry value, i.e. $u_a - u_w = 50$ kPa, of the reconstituted CDT, the shear wave velocities

increase significantly. The soil specimens behave as were essentially saturated at a matric suction below

the air-entry value. An increase in the matric suction can be considered as equivalent to an increase in 150 200 250 300 350 400 0 50 100 150 200 Matric suction, $u_a - u_w$ (kPa) 150 200 250 300 350 400 0 50 100 150 200 Matric suction, $u_a - u_w$ (kPa) (b) (a) $p - u_a = 200$ kPa $p - u_a = 110$ kPa $p - u_a = 300$ kPa $p - u_a = 400$ kPa $p - u_a = 500$ kPa $p - u_a = 200$ kPa $p - u_a = 110$ kPa $p - u_a = 300$ kPa $p - u_a = 400$ kPa $p - u_a = 500$ kPa S h e a r w a v e v e l o c i t y , $v_s(vh)$ (m / s) S h e a r w a v e v e l o c i t y , $v_s(hv)$ (m / s) 150 200 250 300 350 400 0 50 100 150 200 Matric suction, $u_a - u_w$ (kPa) (c) $p - u_a = 200$ kPa $p - u_a = 110$ kPa $p - u_a = 300$ kPa $p - u_a = 400$ kPa $p - u_a = 500$ kPa S h e a r w a v e v e l o c i t y , $v_s(hh)$ (m / s)

Figure 15. The shear wave velocity, (a) $v_s(vh)$, (b) $v_s(hv)$ and (c) $v_s(hh)$, plotted against matric suction (Ng & Yung

2005).

the mean effective stress, and hence the shear wave velocities increase (Mancuso et al. 2000). The rate

of increase of the shear wave velocities reduces as the matric suction exceeds the air-entry value of the

material ($u_a - u_w = 50$ kPa). Air-water menisci (or contractile skins) form at the inter-particle contacts

as the soil specimens start to desaturate at matric suctions exceeding the air-entry value. This results in

an increase in the normal force acting on the particles and thus the shear wave velocities increase. Upon 1.0 1.1 1.2 1.3 1.4 1.5 1.6 1.7 1.8 0 50 100 150 200 250 300 350 400 450 500 Net mean stress, $p - u_a$ (kPa) G_{hh} / G_{hv} $u_a - u_w = 50$ kPa $u_a - u_w = 100$ kPa $u_a - u_w = 200$ kPa $u_a - u_w = 0$ kPa $u_a - u_w = 100$ kPa Reconstituted specimens: Block specimens: $u_a - u_w = 0$ kPa (Ng et al 2004b) $u_a - u_w = 0$ kPa

Figure 16. Degree of stiffness anisotropy of intact and reconstituted specimens of CDT under isotropic net mean

stress (Ng & Yung 2005).

further increase of the matric suction, the meniscus radius reduces, which limits the increase of the shear

wave velocities (Mancuso et al. 2000).

Figure 16 shows the degree of stiffness anisotropy, expressed as G_{hh} / G_{hv} , of reconstituted specimens

of CDT under isotropic net mean stresses at various values of matric suctions. The measured degrees of

stiffness anisotropy of the intact specimens of CDT at matric suctions of 0 and 100 kPa are also shown in

the figure for comparison. The degree of stiffness anisotropy of the reconstituted and intact specimens

appears not to be affected by the matric suction. Over the range of the applied net mean stress, the degree

of stiffness anisotropy is essentially constant in both types of specimen. The anisotropy of the soil fabric

is probably not altered by the net mean stress over the range of the net mean stress considered. The recon

stituted specimens consistently show a lower degree of stiffness anisotropy ($G_{hh} / G_{hv} = 1.0 - 1.1$) than

do the intact specimens ($G_{hh}/G_{hv} = 1.4 - 1.5$). The anisotropic soil fabric of natural CDT is destroyed

in the reconstituted specimens, resulting in similar shear moduli in the horizontal and vertical plane.

7.5 Effects of cyclic loading and creep on the small-strain stiffness of decomposed tuff and rhyolite

Undrained cyclic triaxial compression tests with local strain measurements were conducted on Mazier

specimens of completely decomposed tuff (CDT) from Luen Wo Hui taken at a depth of 29 m and

completely decomposed rhyolite (CDR) from Site A taken at a depth of 27 m (Fig. 1). Figure 17 shows

the stress paths throughout the tests. Back-pressure saturation and isotropic consolidation were car

ried out under an isotropic effective stress, which is equivalent to the in situ lateral effective stress

($p' = 118$ kPa; $q = 0$ kPa). The specimens were then anisotropically consolidated to the in situ effective

stress ($p' = 170$ kPa; $q = 156$ kPa). After the anisotropic consolidation was completed, the specimens

were anisotropically consolidated to an effective stress state of $p' = 548$ kPa and $q = 504$ kPa, which was

a foundation stress state. The stress state was maintained for 10 hours such that the axial strain rate was

maintained at less than 0.002% per hour.

An undrained cyclic triaxial test of 100 cycles starting with compression was carried out after con

solidation, with a cyclic loading amplitude (σ_v) of ± 153 kPa and a loading frequency of one cycle per

minute (0.0167 Hz). The undrained secant Young's modulus (E_u) of the materials during cyclic loading

was determined and the values of E_u during the first compression in the cyclic loading (A to B in Fig. 17)

and the first recompression in the cyclic loading (C to D in Fig. 17) are presented in this paper. After

cyclic loading, a period of 11 hours was allowed for the specimens to creep until the stress state returned 0 100 200 300 400 500 600 700 800 900 1000 0 100 200 300 400 500 600 p' (kPa) q (k P a) Anisotropic consolidation to in situ effective stress Anisotropic consolidation to foundation loading stress state Undrained cyclic loading (100 cycles) Creep Undrained compression A B C D E

Figure 17. Stress paths of the tests on CDT and CDR (Mazier specimen).

to A in Figure 17 and the axial strain rate was maintained less than 0.002% per hour. The specimens

were then sheared under undrained compression at a rate of 0.2% per hour. The undrained secantYoung's

modulus during undrained compression is presented (A to E) in Figure 17. Figure 18 shows the undrained secantYoung's modulus (E_u) of the materials at various stages normal

ized by the initial mean effective stress (p'_{θ}). The normalized undrained secant Young's moduli (E_u / p'_{θ})

at three stages are presented: (a) first compression in the cyclic loading (A to B in Fig. 17); (b) first

recompression in the cyclic loading (C to D in Fig. 17); and (c) compression after cyclic loading and

creep (A to E in Fig. 17). The normalized undrained secant Young's moduli (E_u / p'_{θ}) decrease as axial

strains increase over the range of the measured axial strains in all three stages. The undrained secant

Young's modulus of CDR is slightly higher than that of CDT at each respective stage. The rate ofYoung's

modulus degradation is different at different stages of undrained compression for both materials. Both

materials show the lowest stiffness in the first compression of the cyclic loading (A to B). After sub

sequent unloading, the materials show the highest stiffness along the recompression loading path (C to

D), where $E_{u/p'}^0$ increases by about 100% compared with the first compression loading (A to B) for

both materials. A 180° stress path rotation is involved in the recompression, resulting in the stiffest

response of the materials, which demonstrates the effect of the recent stress history on decomposed

materials (Wang & Ng 2005). The undrained Young's modulus of CDR and CDT at an axial strain of

0.01% during the undrained compression after cyclic loading and creep (A to E) increased by about 30%

and 15%, respectively, compared with the undrained Young's modulus of the materials determined in

the first compression of the cyclic loading (A to B). As time was allowed for the specimens to creep,

deformation of the specimens was reduced to a rate that was not significant in affecting subsequent

stiffness measurements. The increase in the undrained Young's modulus in the materials after cyclic

loading and creep (A to E) may be due to the expansion of yield surface of the materials during the first

compression of the cyclic loading (A to B).

7.6 Comparison of the small-strain characteristics of naturally decomposed granite and tuff

7.6.1 Shear modulus at very small strains

Figure 19 shows the shear modulus of CDG and CDT in the vertical plane, G_{vh} , determined from shear

wave velocity measurements using bender elements. The shear modulus of the Mazier specimens of CDG 0 200 400 600 800

1000

1200

1400 0.001 0.01 0.1 1 Axial strain, ϵ_a (%) CDT - 1st loading (A-B); $p'_0 = 548$ kPa Reloading (C-D); $p'_0 =$

488 kPa After cyclic loading and creep (A-E); $p'_{\theta} = 548$ kPa

E_u

$/p_{\theta}$

p'_{θ} CDR - 1st loading (A-B); $p'_{\theta} = 548$ kPa Reloading (C-D); $p'_{\theta} = 474$ kPa After cyclic loading and creep (A-E); $p'_{\theta} = 548$ kPa

Figure 18. Normalized undrained secant Young's modulus (E_u / p'_{θ}) of CDT and CDR. $10 \ 100 \ 1000 \ 1000 \ 10000 \ 100000 \ 1000000 \ \sigma_v \text{ 'x } \sigma_h \text{ ' (kPa } 2 \text{) } G_{vh} \text{ (M P a) } \text{ CDG Dept h = 23 m Dept h = 27 m Dept h = 28 m Dept h = 37 m Dept h = 45 m Dept h = 54 m CDT Dept h = 14 m Dept h = 29 m}$

Figure 19. Comparison of G_{vh} of CDG and CDT (Mazier specimens).

(from Yen Chow Street) at depths from 23 m to 54 m is shown. There is an increase in G_{vh} in specimens

taken at greater depths (Depth = 45 m and 54 m), which is related to the lower degree of decomposition

at deeper levels. The values of G_{vh} of the Mazier specimens of CDG taken at depths of 45 m and 54 m are

22% and 50%, respectively, higher than that taken at a depth of 28.5 m under the same effective stress $10 \ 100 \ 1000 \ 1000 \ 10000 \ 100000 \ 1000000 \ \sigma_i \text{ 'x } \sigma_j \text{ ' (kPa } 2 \text{) } G_{ij} \text{ (M P a) } \text{ CDG CDT } G_{hh} \ G_{hv} \ G_{hh} \ G_{hv}$

Figure 20. Comparison of the stiffness anisotropy of CDG and CDT (Mazier specimens).

level. The figure also shows the values of G_{vh} of the Mazier specimens of CDT taken at depths of 14 m

and 29 m. At a similar effective stress level, the value of G_{vh} of the Mazier specimen of CDT at a depth

of 29 m is 64% higher than that at a depth of 14 m, which reflects the lower degree of decomposition at

a greater depth. At a similar depth, the values of G_{vh} of the Mazier specimens of CDT (Depth = 29 m)

and CDG (Depth = 28.5 m) are similar. There appears to be

no obvious difference in the measured shear

modulus at very small strains between CDG and CDT using Mazier specimens. The stiffness anisotropies of the Mazier specimens of CDG and CDT are compared in Figure 20. The

figure shows the shear moduli in the horizontal and vertical planes, G_{hh} and G_{hv} , determined from multi

directional shear wave velocity measurements on Mazier specimens using bender elements. The Mazier

specimen of CDT clearly show stiffness anisotropy, with the shear modulus in the horizontal plane higher

than that in the vertical plane ($G_{hh} > G_{hv}$). The stiffness anisotropy is attributed to the horizontal layering

structure of CDT as a result of metamorphism in the parent rock, which is discussed in Section 7.3.1.

Stiffness anisotropy is apparently not obvious in the Mazier specimen of CDG as revealed in the single

set of laboratory test data. Disturbance during Mazier sampling may damage the anisotropic structure

of CDG if such a soil structure exists. Further tests are required to study the shear modulus of CDG in

different planes using specimens with the least degree of sample disturbance (e.g. block specimens).

7.6.2 Variations in shear modulus with shear strain

Constant p' triaxial compression and extension tests were performed on Mazier specimens of CDT in

a triaxial apparatus equipped with bender elements and local strain transducers (Fig. 9a). Shear wave

velocity measurements were taken before constant p' shearing to determine the shear modulus at very

small strains. The compression tests and the extension tests involve different angles of stress path rotation

(the angle measured anti-clockwise from the approaching stress path). The angle of stress path rotation

in the compression tests was 90° , while that in the extension tests was 135° . Figures 21a and 21b show

the normalized shear modulus degradation curves (G_{sec}/G_{vh} vs. ϵ_q) of CDT along the compression and

extension paths, respectively. The test data for CDG are also shown in the figures for comparison. The variations in the shear modulus of CDT with deviatoric strain are generally consistent with those

of CDG along both compression and extension paths. The data from the tests on CDT lie close to the

upper bound of the test data from CDG along both compression and extension paths. Fine materials (e.g.,

clays) exhibit less shear modulus degradation at a given strain level compared with coarse materials (e.g.,

sands) (Stokoe et al. 1995). CDT is slightly finer than CDG (Fig. 4), but the difference in particle size

is not so significant to result in an obvious change in the shear modulus degradation curve. Along the compression path ($\theta = 90^\circ$; Fig. 21a), the normalized shear modulus of CDT reduces by

60% at a deviatoric strain of 0.01%. The shear modulus of CDT is higher along the extension path

($\theta = 135^\circ$; Fig. 21b), where the normalized shear modulus reduces by 30% at a deviatoric strain of 0.01%.
 0.0 0.1 0.2 0.3 0.4 0.5 0.6 0.7 0.8 0.9 1.0 0.001 0.01 0.1 1 Deviatoric strain, ϵ_q (%) G_{sec}/G_{vh} CDT CDG
 0.0 0.1 0.2 0.3 0.4 0.5 0.6 0.7 0.8 0.9 1.0 0.001 0.01 0.1 1 Deviatoric strain, ϵ_q (%) G_{sec}/G_{vh} CDT CDG (b) (a) $p' = 400$ kPa; $\theta = 90^\circ$ $p' = 100$ kPa; $\theta = 90^\circ$ $p' = 150$ kPa; $\theta = 90^\circ$ $p' = 200$ kPa; $\theta = 90^\circ$ $p' = 250$ kPa; $\theta = 90^\circ$ $p' = 300$ kPa; $\theta = 90^\circ$ $p' = 400$ kPa; $\theta = 135^\circ$ $p' = 100$ kPa; $\theta \approx 180^\circ$ $p' = 150$ kPa; $\theta \approx 180^\circ$ $p' = 200$ kPa; $\theta \approx 180^\circ$ $p' = 250$ kPa; $\theta \approx 180^\circ$ $p' = 300$ kPa; $\theta \approx 180^\circ$

Figure 21. G_{sec}/G_{vh} vs. ϵ_q of CDG and CDT (Mazier specimen): (a) Compression tests; (b) Extension tests.

0.01%. The difference in the measured normalized shear modulus is clearly shown until the deviatoric

strain reaches about 1%. Atkinson et al. (1990a) suggested that the change of subsequent stiffness as

a result of either a sudden change in the direction of the stress path or a period of time at a constant

stress state is the effect of recent stress history. The increase in soil stiffness with increasing angles of

stress path rotation shown in Figures 21a and 21b is the effect of the recent stress history as a result

of changes in the direction of the stress path. Similar behaviour is also observed in other types of soil.

Atkinson et al. (1990a) showed that the shear modulus of reconstituted London Clay increases up to

one order of magnitude (at a deviatoric strain of about 0.01%) with increasing magnitude of θ and that

the effect of stress path rotation vanishes when deviatoric strain reaches about 0.5%. Hird & Pierpoint

(1997) and Pourie et al. (1998) showed that the shear modulus of clay is higher after a 180° stress

path rotation. Amorosi et al. (1999) and Clayton & Heymann (2001) demonstrated the difference in soil

stiffness derived from compression and extension tests due to the recent consolidation stress history. Table 8. Void ratio changes of CDT resulting from reconsolidation to in situ stresses. Specimen p' (kPa) q (kPa) $\Delta e/e$ ϵ_v (%) ϵ_a (%) ϵ_r (%) B_{aniso} 80 80 -0.01 -0.2 +1.1 -0.6 M_{aniso} 100 100 -0.01 -0.4 +3.0 -1.7 ϵ_v : volumetric strain. ϵ_a : axial strain. ϵ_r : radial strain.

7.7 Sample disturbance - comparison of Mazier and block sampling

Sampling induces irrecoverable strains in natural soil samples. The soil structure may be damaged

significantly during the sampling process. Soil structure has been shown to have significant effects on

the mechanical properties of soils such as compression characteristics (Burland 1990), strength (Burland

1990, Clayton et al. 1992), stiffness (Clayton et al. 1992, Cuccovillo & Coop 1997), yielding and large

strain behaviour (Cuccovillo & Coop 1999). It has been shown that different sampling techniques result

in different degrees of sample disturbance in terms of the magnitude of induced strains and the reduction

of the mean effective stress and shear wave velocity (Baligh et al. 1987, Hight 1993, Lo Presti et al.

1999, Lohani et al. 1999). It is therefore important to assess the effect of sampling when determining

the mechanical properties of natural soils. The degree of sample disturbance can be assessed by the amount of volume change resulting from

reconsolidation of a soil specimen to the estimated in situ effective stresses (Hight 1993, Tanaka et al.

1996, Lunne et al. 1997, Lo Presti et al. 1999). Table 8 shows the measured void ratio change and

strains resulting from the reconsolidation of the block (B_{aniso}) and Mazier (M_{aniso}) specimens of

CDT to the estimated in situ effective stresses. The void ratio change of the block specimen resulting

from reconsolidation is small ($\Delta e/e_0 = 0.01$). According to the criterion proposed by Lunne et al. (1997)

for evaluating sample disturbance, the quality of a soil sample is considered very good to excellent if

e/e_0 is less than 0.04 for soils with overconsolidation ratios between 1 and 2. Although the void ratio

change of the Mazier specimen is also small ($\Delta e/e_0 = 0.01$), the associated axial and radial strains are

relatively large. Sample disturbance in the Mazier specimen is believed to be larger than that in the block

specimen. A higher degree of sample disturbance is responsible for the lower shear moduli and degree of stiffness

anisotropy of the Mazier specimens. It is shown in Section 7.3.1 that the Mazier specimen shows a

lower degree of anisotropy ($G_{hh}/G_{hv} = 1.36$) compared with the block specimen ($G_{hh}/G_{hv} = 1.48$). The

anisotropic structure of CDT may be damaged by the tube sampling process, resulting in a lower degree

of anisotropy in the Mazier specimen. The shear moduli of the Mazier specimen are also lower than that

of the block specimen as shown in Sections 7.3.1 and 7.3.2. The shear moduli, G_{hh} , G_{hv} and G_{vh} , of the

Mazier specimen are 72%, 58% and 81% lower than those of the block specimen, respectively (Fig. 14).

These differences correspond to the decrease in shear wave velocities, $v_s(hh)$, $v_s(hv)$ and $v_s(vh)$, by 27%,

21% and 29%, respectively, and a decrease of bulk modulus by 7% in the Mazier specimen.

8 SUMMARY AND CONCLUSIONS

The small-strain stiffness of completely decomposed granite (CDG), tuff (CDT) and rhyolite (CDR)

at four test sites in Hong Kong was studied through field and laboratory measurements. Field testing

techniques including suspension P-S velocity logging, crosshole seismic measurements and self-boring

pressuremeter (SBPM) tests were conducted on decomposed granite to study the shear moduli of the

material at small strains. Laboratory testing techniques were developed, which enabled multidirectional

shear wave velocity measurements using bender elements and control of matric suction in a triaxial

apparatus with local strain measurements. Various aspects of soil stiffness at small strains were studied,

including the variations in shear modulus with increasing shear strains (CDG and CDT), the stiffness

anisotropy at very small strains (CDG and CDT), the effects of matric suction on stiffness anisotropy

(CDT), the effects of cyclic loading and creep on small-strain stiffness (CDT and CDR), and the effects

of sample disturbance on shear modulus and stiffness anisotropy (CDG and CDT). In the field, similar shear wave velocities of decomposed granite were measured by the suspension P-S

velocity logging and crosshole seismic measurements conducted at two sites with similar soil profiles.

The measured shear wave velocities are consistent with the soil profiles, where both the shear wave

velocity and the SPT 'N' value increase with depth and the measured shear wave velocity increases as

the material is subjected to less weathering at a deeper level. The shear wave velocity ($v_s(vh)$ or $v_s(hv)$) and

shear modulus (G_{vh} or G_{hv}) of completely decomposed granite (CDG) varies from 200 m/s to 350 m/s

and 45 MPa to 280 MPa, respectively, across a depth of about 20 m to 40 m. An empirical correlation

between the shear modulus and the SPT 'N' value of CDG ($G_{vh} = 14.3 \times N^{0.68}$) was established and was

found to be similar to that proposed by Imai & Tonouchi (1982) for sands. The SBPM tests conducted at

two sites show consistent results, where the stiffness-strain relationship is highly non-linear with G_{sec}/p'

decreasing from about 300 to about 50 as shear strain increases from 0.02% to 1%.

Laboratory measurements of the shear modulus (G_{vh}) of the Mazier specimens of CDG show a 20%

to 50% reduction compared with that measured in the field. The lower shear modulus measured in the

laboratory may be attributed to sample disturbance. At the same effective stress level, the measured G_{vh}

in Mazier specimens taken at a greater depth is higher than that at a shallower depth. For instance, the

value of G_{vh} of the Mazier specimens of CDG taken at a depth of 54 m is 50% higher than that taken at a

depth of 28.5 m when tested at the same effective stress level, while a 64% increase in G_{vh} is observed in

the Mazier specimen of CDT taken at a depth of 29 m compared with that at a depth of 14 m. The higher

G_{vh} in specimens taken at a deeper level reflects a lower degree of decomposition of the materials at a

greater depth. The measured shear modulus (G_{vh}) of the Mazier specimens of CDG at very small strains

is similar to that of CDT for Mazier specimens taken at similar depths and tested under similar effective

confining stresses.

Constant p' triaxial compression and extension tests on Mazier specimens of CDG and CDT showed

that the materials are highly non-linear and the normalized shear modulus (G_{sec}/G_{vh}) is higher along the

extension path (angle of stress path rotation, $\theta > 90^\circ$) than along the compression path ($\theta \leq 90^\circ$). For

CDG, the value of G_{sec}/G_{vh} along the extension path is about 60% higher than that along the compression

path at a deviatoric strain of 0.01% and the difference diminishes as deviatoric strain increases. For CDT,

the value of G_{sec}/G_{vh} is higher along the extension path than along the compression path by about 75%

at a deviatoric strain of 0.01%. The shear modulus of CDT is higher along the extension path ($\theta = 135^\circ$),

where the normalized shear modulus reduces by 30% at a deviatoric strain of 0.01%. The difference

in the stiffness of the materials along the different paths is related to the effect of the recent stress

history through which soils show a stiffer response as the angle of the stress path rotation increases. The

variations in the normalized shear modulus (G_{sec}/G_{vh}) of CDG and CDT with deviatoric strain (ϵ_q) are

generally consistent along both compression and extension paths, with the stiffness-strain relationship

of CDT lying close to the upper bound of that of CDG.

The effect of the recent stress history on the small-strain stiffness of saprolites is also exhibited during

the undrained cyclic triaxial compression tests. The Mazier specimens of CDT and CDR show the highest

stiffness along the recompression loading path, where $E_u/p'_{0\max}$ increases by about 100% compared with

the first compression loading. A 180° stress path rotation is involved in the recompression, resulting in

the stiffest response of the materials, which demonstrates the effect of the recent stress history on the

small-strain stiffness.

Multidirectional shear wave velocity measurements revealed the anisotropic characteristic of CDT.

The degrees of inherent anisotropy (G_{hh}/G_{hv}) determined at the isotropic stress state ($K = 1.0$) are 1.48

and 1.36 for the block and Mazier specimens, respectively. A higher shear modulus in the horizontal plane

is attributed to the layering structure in the horizontal plane resulting from metamorphism of the parent

rock. The lower degree of inherent stiffness anisotropy of the Mazier specimen is due to a higher degree

of sample disturbance in the Mazier specimen. At the anisotropic stress state ($K = 0.4$), the degrees of

anisotropy (G_{hh}/G_{hv}) reduced to 1.26 and 1.15 in the block and Mazier specimens, respectively, which

is a 15% reduction from the measured inherent anisotropy due to the stress-induced effect. The effect

of a stronger structure in the horizontal plane on the shear moduli (i.e. $G_{hh} > G_{hv}$) prevails even under a

higher vertical effective stress condition ($K = 0.4$), which is unexpected with no prior knowledge of the

anisotropic structure of the material. The shear moduli (G_{hh} , G_{hv} and G_{vh}) of the Mazier specimen of CDT

are on average 70% lower than those of the block specimen due to a higher degree of sample disturbance.

Preliminary measurements of G_{hh} and G_{hv} of the Mazier specimen of CDG suggested that the degree

of anisotropy of CDG, if any, is lower compared with the Mazier specimens of CDT. Further tests are

required to study the stiffness anisotropy of CDG using specimens with the least sample disturbance

(e.g., block specimens).

The shear wave velocities, $v_s(vh)$, $v_s(hv)$ and $v_s(hh)$, of unsaturated reconstituted CDT depend on the two

stress state variables, matric suction ($u_a - u_w$) and net mean stress ($p - u_a$). The shear wave velocities

increase non-linearly with an increase in matric suction at a constant net mean stress. The rate of increase

in the shear wave velocities reduces when the applied matric suction exceeds the air-entry value of the

soil. The degree of stiffness anisotropy of the reconstituted and intact specimens is not sensitive to the

matric suction for the range of matric suction considered ($u_a - u_w = 0$ to 200 kPa).

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Weathered granite in Japan

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ABSTRACT: In the western parts of Japan, steep mountain slopes and hills in the vicinity of urban

cities are often composed of weathered granite and these slopes fail due to heavy rains or typhoons. Also

the abundance of weathered granite nearby urban cities has prompted the use of weathered granite as

reclaimed fill materials or foundation materials to support numerous infrastructure facilities. The Great

Hanshin-Awaji Earthquake of 1995, however, has revealed the vulnerabilities of weathered granite

fill materials under intense dynamic loadings. Thus, the weathered granite has brought about various

geotechnical problems and led to research activities on its properties both in the natural state and in the

remolded state or as a construction material. In this paper, the geotechnical engineering properties of

weathered granite are discussed by broadly dividing into two parts: first, the properties in undisturbed

state and second, the properties in disturbed state or as a construction material. After describing the

distributions and origins of the weathered granite in Japan, the classifications of weathering, sampling

methods of undisturbed specimens and mechanical properties in natural states are discussed. Then the

dynamic properties of weathered granite are discussed in view of its liquefaction strength properties.

1 INTRODUCTION

The distribution of granite in Japan is shown in Fig. 1 (Takahashi, 1999), and the total area of granite

deposit is about 13% of total surface area of Japan. It is obvious that the majority of granite is found in

the west-northern section of Japan that is bounded by two major faults, the Itoigawa-Shizuoka Tectonic

Line and the Japan Median Tectonic Line (MTL). The other granite deposit is found in the northern half

of Japan Honshu Island. The Itoigawa-Shizuoka Tectonic Line defines the west edge of the tectonically

active narrow zone (so called Fossa Magna) that separates the main Japan Island into the eastern and the

western sections. This zone is created by a complex tectonic movement of the two oceanic plates; the

Pacific and the Philippine Plates. The north-eastern Japan Honshu Island is created by the subduction of

the Pacific Plate below the Eurasian Plate, which is a continental plate. The subduction of the Philippine

Plate below the Eurasian Plate has created the western part of Japan Honshu Island with the MTL

representing a major feature of tectonic movement in that area. Figure 2 illustrates how these oceanic

plates subduct beneath the continental plate of the Eurasian Plate.

The formation of granite is closely associated with the tectonic interaction between the oceanic plate

and continental plate. Granite is an igneous rock that has cooled slowly within the crust of the Earth, and

its formation occurred due to the intrusion of hot magma into the continental crust, which in this case

is the Eurasian Plate, from the plate interface below. Consequently the granite rock has been distributed

along the north-eastern and western sections of Japan Island where the Philippine and Pacific Plates

subduct beneath the Eurasian Plate.

The geological time for these formations varies from the Cretaceous (135 to 65 million years ago) to

Paleogene (65 to 23 million years) periods for the Hiroshima (Sanyo) and Ryoke Granites in western

Honshu Island and the Cretaceous period for the Kitagami Mountains in northern Honshu Island. Granite

with its origin older than the Cretaceous period occupies a very small area in Hokuriku Region (located

northwest side of Itoigawa-Shizuoka Tectonic Line). Hiroshima (Sanyo) and Ryoke Granites cover a large

area of the western section of Japan Island. Weathering of Hiroshima (Sanyo) and Ryoke Granites are

well known and there have been many geotechnical problems associated with these weathered granites.

Figure 1. Distribution of granite in Japan. Figure 2. Plate tectonics along south of Japan.

2 GEOTECHNICAL PROBLEMS ASSOCIATED WITH WEATHERED GRANITE

In Japan, the weathered granite is often called Masa-do, or simply Masa, that implies "true sand" because,

when the mother rock disintegrates into individual particles, the sand particle shows its clean whitish

appearance due to high quartz content. The sand beaches along the Seto-Inland Sea (i.e., a narrow straight

between Shikoku Island and West Honshu Island) usually exhibit beautiful white sand beaches with a

sharp contrast of dark blue pine trees. In the engineering world, however, the weathered granite has brought about various geotechnical

problems and led to research activities on its properties both in the natural state and in the remolded

state or as a construction material. In the natural state, steep mountain slopes and hills often fail due

to heavy rains or typhoons, because a high permeability of the weathered granite allows the seepage

flow to quickly reach deeper depths and saturate the sloping ground. There were many large natural

disasters related to rainfall induced landslides between 1960 and 1970, as will be described later. The

abundance of weathered granite in the vicinity of urban cities has prompted the use of weathered granite

as reclaimed fill materials or foundation materials to support numerous infrastructure facilities. Such

geotechnical construction activities were prevalent during the 1960-1970 period, when infrastructure

construction in Japan was expanding rapidly. Between 1970 and 1980, intensive research and publi

cation efforts were focused on the static geotechnical properties of weathered granite. Examples of

these publications are the books published by several research committees of JSSMFE (1974, 1979,

1990). The weathered granite is an easily compactable material with good mechanical properties when prop

erly handled. However, the Great Hanshin-Awaji Earthquake of 1995 in Kobe, Japan, taught engineers

an expensive lesson on the damage caused by liquefaction of reclaimed man-made islands which were

constructed by reclaiming near-shore water fronts with coarse fill materials of decomposed granite.

In addition, there were many landslides on mountain slopes during the Great Hanshin-Awaji Earth

quake. After the earthquake, there were also many rainfall induced slope failures. In the geotechnical

engineering community, the Great Hanshin-Awaji Earthquake marks an important turning point in

research activities, as a result of which more attention

was paid to the dynamic behavior of coarse

fill materials (mainly of decomposed granite). For example, efforts have been made to take a large

frozen undisturbed sample of coarse fill material in order to examine the liquefaction strength of such

material. In the following sections, we will discuss the geotechnical engineering properties of weathered granite

by broadly dividing into two parts: first, the properties in undisturbed state and second, the properties in

disturbed state or as a construction material. Table 1. Rainfall induced landslides in weathered granite area 1) 1938 Great Hanshin Rainfall 461.8 mm cumulative over July 3-5 Flood No. of slides 2,727, No. of death: 616 2) 1961 Tenryu 36 Rainfall 579 mm cumulative over June 23-July 1 Disaster No. of slides: numerous, No. of death: 136 3) 1967 Uetsu Heavy Rain Rainfall 748 mm cumulative over Aug. 26-30 Disaster No. of slides: 100 Slides/km² (density), No. of death: 131 4) 1967 Kure Disaster Rainfall 317 mm cumulative over July 7-10 No. of slides: over 1000, No. of death: 88 5) 1967 Rokko Disaster Rainfall 379.4 mm cumulative over July 5-9 No. of slides: 2,549, No. of death: 92 6) 1972 Nishimikawa Rainfall 219 mm cumulative on July 12 (5 hrs.) Disaster No. of slides: 3,000, No. of death: 64 7) 1988 Hiroshima Rainfall 264 mm cumulative in July Disaster No. of slides: over 200, No. of death: 14 8) 1999 Hiroshima Rainfall 271 mm cumulative on June 29 Disaster No. of slides: over 200, No. of death 32 (20 within the city)

Figure 3. Location of major landslides in weathered granite.

3 STATIC GEOTECHNICAL PROBLEMS OF DECOMPOSED GRANITE IN

NATURAL STATE

Historically, the studies on decomposed granite in Japan have been primarily motivated by many rainfall

induced landslides in the weathered granite mountainous areas that are close to urban built environments.

Chigira (2002) has described examples of such rainfall induced major landslides in the past in the

weathered granite area together with the number of casualties and the intensity of rainfall as shown

on Table 1 and in Fig. 3 for their locations. As can be seen from Fig. 3, the frequent occurrence of

landslides is located in a narrow belt zone, just north of MTL fault, and the weathered granite in this Table 2. Classification of weathered granite (JSSMFE 1979). Pressure meter test Porosity Wet density Soil Description E(kgf/cm²) n(%) p t (g/cm³) D L Crashes into fine 50-300 35-43 1.9 powders under finger press D M Crashes into sand 300-800 20-35 2.1 (quartz) particles under finger press D H May crash into 800-1500 14-20 2.2 sand or gravels under hard finger

zone is classified as Ryoke Granite. As such, the weathered granite in the western part of Japan, mainly

the Ryoke Granite and Hiroshima Granite in the vicinity, have been the prime study materials for the

geotechnical studies. Table 1 also shows that a large concentration of catastrophic landslides occurred

in the 1960-1970 period, which is why geotechnical studies on the undisturbed weathered granite of

natural state were carried out intensively during these periods. Because of such catastrophic landslides associated with intensive rainfalls, there were many studies

on how to predict an imminent of landslide or provide an early warning for evacuation of the residents

in the rainfall induced landslide prone area. For example, Aboshi (1972) proposed a chart based on the

relationship between the cumulative rainfall in previous two weeks and the present rainfall intensity. The

rainfall induced landslide is predicted to occur when these two parameters exceed certain limits. The following sections provide a summary of previous Japanese studies on engineering properties of

undisturbed weathered granite in its natural state, with an emphasis on three main areas: Weathering

classification, Undisturbed sampling, and Strength and Deformation characteristics.

3.1 Classification of weathered granite

Various classification systems are available in Japan for describing the soundness of rock. These clas

sification systems were developed in response to the different design requirements for the foundations

of different engineering structures, such as tunnels, dams, or bridges. Among the rock classification

systems in Japan, the most relevant system for the weathered granite would be the classification system

proposed by Honshu-Shikoku Bridge Authority. The classification is a derivative of the system that was

proposed by Tanaka (1964), which described the rock from very sound to very poor in six levels of

classification (A, B, C H , C M , C L , and D). The classification level relevant to the weathered granite is the

lowest level "D", and this level has been further subdivided into three levels, D H , D M , and D L . The difference among the three levels of weathering; D H , D M , and D L may be illustrated in Table 2,

which shows the variations among the deformation and physical properties such as porosity and density. The classification given in Table 2 is mostly based on the visual inspection of granite samples and may

be somewhat ambiguous. However, there are positive correlations among the physical and mechanical

properties, which have been examined by various researchers. For example, Nishida and Aoyama (1979

and 1984) used the void ratio as an index property to measure the level of weathering, and demonstrated

that the tensile strength, which is a mechanical property sharply reflecting the degree of weathering,

changes with the void ratio in a consistent manner as shown in Fig. 4. According to their findings, based

on nearly 30 different weathered granites, the void ratio

increases with the level of weathering. However,

at the highest level of weathering that occurs near the surface of natural sloping ground, the void ratio

of surface soil could decrease due to the re-compaction associated with the structural collapse of highly

weathered soil skeleton, as noted by Lumb (1962). The reason for the increase in tensile strength with

soils of higher void ratio is that, as the level of weathering increases, the soil particles disintegrate into

more clayey particles that in turn increases the soil capacity to resist tensile forces. Nishida and Aoyama also examined the ignition loss of various weathered granites, using the void

ratio as a weathering index parameter. The relationship between the ignition loss and the void ratio is

shown in Fig. 5, in which the void ratio increases with ignition loss up to a point, following which it

decreases with increasing ignition loss. Such a trend is consistent with the re-compaction phenomena

near the surface of natural sloping ground. Their preference for using void ratio, instead of ignition loss,

as an index parameter is that the measured range in the void ratio covers nearly all phases of weathering

Figure 4. Relationship between tensile strength and void ratio.

Figure 5. Relationship between ignition loss and

void ratio. Figure 6. Relationship between ignition loss and porosity.

from the modest to the highest, while the ignition loss is only useful for highly weathered granite and is

difficult to measure for the modestly weathered granite.

Figure 6 illustrates another example of the relationship between the porosity and the ignition loss for

highly weathered granite as reported by Onodera et al. (1976). The test data is based on the Ryoke Granite

in Shikoku Island. At this site, there is a good correlation between the ignition loss and the porosity.

Other similar research on the mechanical properties of weathered granite, which adopted the ignition

Figure 7. Undisturbed sampling & specimen

preparation using soil nailing. Figure 8. Core bits to trim 5 cm dia. specimen from frozen block sample.

loss as an index parameter for weathering, will be presented in the following section of Strength &

Deformation Properties. Nishida and Aoyama (1984) and Nishida (1990) discussed the classification system of weathered

granite, and pointed out that the classification system could vary considerably due to differences in the

mother granite rock, as well as the various weathering processes associated with the different topographic

and geomorphological features of sloping ground. There would be a wide variety of weathering profiles,

such as those with heavy weathering near the ground surface and modest weathering at considerable

depths, due to different climatic, topographic and geomorphological conditions. Nishida (1990) listed 10

different classifications that have been proposed by Japanese researchers and pointed out the difficulty

of having a universally applicable classification system. The classification of weathering by using the

mechanical properties is also difficult, because the sampling and testing of undisturbed weathered granite

is extremely difficult.

3.2 Undisturbed sampling of decomposed granite

The sampling of undisturbed weathered granite has been

attempted by various researchers using different techniques. The most popular method of undisturbed sampling is to perform block sampling in the field and cut out the test specimens from the block sample. Nishida and Aoyama (1981) used a combination of soil nailing and cutting mold to obtain a large size specimen for large direct shear testing. Figure 7 depicts their sampling and specimen preparation method. In the field, a square plate with perimeter holes for soil nailing is placed over a flat surface of weathered granite. By nailing many 150 mm long steel nails along the perimeter (of a 180 mm diameter circle in the figure), a large size block sample can be taken out from the natural ground. Later, in the laboratory, the block sample is placed in an up-side down position to trim out a large test specimen using a cutting mold as shown in the figure. For obtaining large size specimens, it may be possible to trim out the specimens carefully from the block sample. However, it is very difficult to obtain smaller size triaxial specimens from the block sample. Yagi and Yatabe (1985) and Murata et al. (1987) used specially designed core bits to trim out small triaxial specimens from frozen block samples. Figure 8 shows their core bits which were specially designed to minimize the heat generated, and to flush out the cuttings during trimming. Yagi and Yatabe (1985), for example, succeeded in taking as many as 16 specimens of 5 cm diameter from a 30 cm × 30 cm block sample.

3.3 Strength & deformation properties

3.3.1 Effects of weathering on the strength & deformation properties

The changes in strength and deformation properties with different degrees of weathering have been

studied by various researchers using undisturbed granite soils, for example Onodera et al. (1976),

Figure 9. Triaxial compression test results of undisturbed weathered granite specimen.

Murata et al (1987) and Shimizu (1990). Murata et al (1987) examined the variation of deformation

and strength properties of weathered granite using 12 different samples taken from the Hiroshima and

Yamaguchi areas. They used the ignition loss for evaluating the level of weathering, but suggested a

modified procedure for measuring the loss. In the Japan Geotechnical Society standards, the ignition

loss is measured for soil particles (weight of 2 g) finer than 0.42 mm. Murata et al. used particle sizes

taken from the field with a weight of 5 g. By modifying the procedure, they demonstrated that the

resulting correlations with the ignition loss are improved. Figure 9 shows their triaxial test results, in

which consistent changes were obtained in both the measured strength and deformation characteristics.

It should be noted that two values of ignition loss are listed in the figures, with the one in parentheses

based on the standard JGS procedure.

It is clear that the level of weathering affects the strength and deformation properties significantly. The

peak strength is higher for the less weathered granite, which also exhibits a stiffer deformation response.

The dilatancy characteristic is also different among the specimens, with the highly weathered material

showing more contractive behaviour.

Shimizu (1990) examined three different weathered granites

in Tottori, Okayama and Kagawa pre

fectures and presented the changes in the strength and deformation properties by adopting the void ratio

as a weathering index parameter. Figure 10 shows the changes in the friction angle with the void ratio.

Although the weathered granite has both cohesive and frictional resistances, the decrease in the fric

tional resistance with weathering is apparent from the figure. Onodera et al (1976) also reported a similar

decrease in the friction angle with the increase in weathering, but showed that the cohesive parameter,

c' , does not decrease significantly with weathering. Murata et al (1987) also obtained a similar result

on the relationship among c' , ϕ' and ignition loss as shown in Fig. 11, in which the cohesive parameter

c' is nearly constant irrespective of the weathering level. Their results also indicate that the cohesive c'

parameter decreases with the saturation of soil.

Figure 10. Relationship between void ratio and

friction angle. Figure 11. Relationship between ignition loss and friction angle.

Figure 12. Relationship between void ratio and

normalized shear modulus. Figure 13. Relationship between void ratio and compression index. The effect of weathering on the deformation properties was also examined by Shimizu (1990). The

variations of shear modulus and compression index with the level of weathering are examined and shown

on Figs. 12 and 13. In calculating the shear modulus, the deformation data for elastic response in the small

strain regime was used. These figures illustrate that the deformation characteristics are also affected by

the level of weathering, with the lower degree of

weathering yielding stiffer deformation responses.

3.3.2 Effects of disturbance on the strength & deformation properties

It is also noted that the lightly weathered granite has a peak strength but the strength drops rapidly

with the increase of shear strain. The decrease in the strength of lightly weathered granite is mainly

due to particle breakage, and the effect of particle breakage or crushing of granite soil on the strength

has been an additional area of study for weathered granite. Yagi & Yatabe (1985) examined the particle

breakage of both undisturbed and disturbed specimens of weathered granite before and after shearing

in the triaxial test. Figure 14 presents the changes of gradation curves for undisturbed and disturbed

(compacted) specimens. It is apparent that the particle breakage is more dominant for the undisturbed

specimen.

Figure 14. Changes in gradation curve before and after shearing of undisturbed and disturbed weathered granite.

Figure 15. Comparison of stress-strain curves between undisturbed and disturbed weathered granite.

The occurrence of particle crushing during shear and under high confining stress leads to an interesting

observation on the difference in stress-strain behaviour between undisturbed and disturbed (compacted)

weathered granite soil. It is well known, in general for soils, that the stress-strain relationship shows

stiffer response for the undisturbed state than for the disturbed state. However, for the weathered granite,

the soil specimen in undisturbed state has a more unstable soil structure, and therefore the stress-strain

relationship shows a less rigid response in comparison with

the disturbed (compacted state). Murata et al

(1987) compared the stress-strain relationship between undisturbed and disturbed weathered granite

under low confining pressures, while keeping the moisture content and the void ratio of the specimens

to be about the same. Their results, shown on Fig. 15, indicate that while the peak strengths are quite

similar, the disturbed samples exhibit a stiffer stress-strain response.

Yagi and Yatabe (1985) carried out a similar study between the undisturbed and the disturbed specimens

as shown on Fig. 16, and obtained a similar conclusion. It is clear from the figure that the volume changes

during shear is more contractive for the undisturbed state.

The highly crushable nature of weathered granite particles in response to shearing also affects the

strength properties under high confining pressures. As shown previously in Fig. 9, the stress-strain curve

is very much affected by the level of confining stress applied at the time of shear. The strength parameter

therefore varies with the level of confining stress. More importantly, it is necessary to consider the

nonlinear strength failure envelope in order to properly model the deformation and strength behaviour

of undisturbed weathered granite. Murata et al (1987 and 1990) examined the applicability of numerical

soil modeling for predicting the stress-strain behaviour of undisturbed weathered granite.

4 DYNAMIC GEOTECHNICAL PROBLEMS OF DECOMPOSED GRANITE AS

CONSTRUCTION MATERIAL

The mechanical properties of undisturbed weathered granite were studied intensively because there had

been many rainfall induced catastrophic landslides in the areas located in the granite hillside or mountains.

Figure 16. Comparison of stress-strain curves between undisturbed and disturbed weathered granite.

Figure 17. Area map of Kobe and nearshore man-made islands.

The Great Hanshin-Awaji Earthquake of 1995 hit Kobe, where substantial quantities of coastal reclaimed

lands had been constructed using "Masa-do", quarried from the weathered granite borrow pits around

Kobe area. The liquefaction damage to these man-made near-shore islands of Kobe was so devastating,

with extensive lateral flows of quay walls and large settlements of man-made islands. In order to gain a better understanding of the earthquake and its liquefaction damages, the following

section reviews the general features of the Great Hanshin-Awaji Earthquake and the observed liquefaction

damage at the reclaimed lands that were constructed by weathered granite fill materials.

4.1 Liquefaction damages at reclaimed lands due to the Great Hanshin-Awaji Earthquake

The epicenter of the earthquake was located at the depth of 14 km and about 25 km west of Kobe city. The

magnitude of the earthquake was 7.2 on the Richter scale. It occurred at 5:46 am, on January 17th 1995.

The fault along the foothills of the Rokko Mountains, which runs north-east to south-west, ruptured.

This lineament of the Rokko Mountains behind Kobe City is clearly shown in Fig. 17(a). Most of the

earthquake damage and casualties were located south of the Rokko Mountains and concentrated along

a very narrow band of area. Figure 17(b) shows the locations of Kobe Port Island and Rokko Island,

Figure 18. Liquefaction damages at reclaimed islands.

Figure 19. Gradation curves of decomposed granite and crushed mud stone.

which are man-made islands built using borrow materials from the Rokko Mountains and surroundings.

The first phase of Kobe Port Island, which is the northern half of the present island, was nearly entirely

built using weathered granite fill material. Rokko Island, on the other hand, was built using a mixture of

weathered granite and crushed mud stones. In Fig. 17(b), the elevation contours of the granite bedrock are

indicated. These man-made islands are constructed over deposits of Pleistocene and Holocene materials

that are 2000 m thick. The water depth in the Kobe Port Island area is about 10 to 15 m. The thickness

of the granite fill material, after allowing for consolidation compression of the superficial soft seabed

deposits, was about 20 to 25 m.

Due to the earthquake, there were extensive liquefaction in these man-made islands. The level of liq

uefaction was more severe on Kobe Port Island than on Rokko Island. Figure 18(a) shows the liquefaction

areas on Kobe Port Island, with average settlement of about 30 cm. It was noted that the deformation

along the perimeter quay wall was much more severe than the settlement of inner fill ground. The perime

ter quay wall was made up of massive concrete caissons placed on a thick bed of weathered granite fill

materials which replaced the superficial soft marine clay as shown in Fig. 18(b). The fill at the base of

the caissons, together with the backfill behind the caissons, liquefied, resulting in extensive lateral flows

and bearing capacity failure of the quay walls. The caissons moved laterally by as much as about 6 m.

4.2 Liquefaction properties of weathered granite fill materials

The gradation curves of fill materials used to reclaim both Kobe Port Island and Rokko Island are

shown in Fig. 19. The fill contained much larger size particles as large as boulders, but the gradations

shown only correspond to the soil samples used for testing. The figure indicates that the fill materials

are well-graded, in contrast to the uniform sand which had been studied extensively in the past for

liquefaction phenomena. Before the 1995 earthquake, most of the evaluation procedures for liquefaction

were established based on the test results on uniform fine sand, such as Toyoura standard sand. For

Figure 20. Compaction curve for decomposed granite and crushed mud stone.

example, the Road Bridges Design Specification in Japan relies very much on the liquefaction strength

data of Toyoura sand. The Great Hanshin-Awaji Earthquake of 1995 has shown that there is a need for a new liquefaction

evaluation procedure for well graded coarse fill materials. Since then, many studies have been made to

identify the liquefaction properties of well graded coarse fill materials, mainly of weathered granite fill

material. In the following, the liquefaction properties that are unique to weathered granite soil will be described.

The effect of fines content on the liquefaction strength, and the procedure for estimating post liquefaction

settlement, will also be discussed in detail.

4.2.1 Liquefaction strength: testing procedure

Because both Kobe Port Island and Rokko Island suffered extensive liquefaction damage, the liquefaction

strength of reclamation fill materials including weathered granite soil and crushed mud stone had to be

studied after the Earthquake. A series of cyclic triaxial tests were performed on these two materials

to obtain the liquefaction strength curves and the post liquefaction settlement properties. The latter

property was examined by performing a re-consolidation test at the end of liquefaction test. The tests

were performed under confining pressures of 1 and 2 kgf/cm². The range of applied confining pressure

corresponds to the overburden stresses at just above the water level and at the bottom of fill layer. The

density of specimen was varied according to both the in-situ compaction state and the compaction test

results. Figure 20 shows the compaction curves for the two materials. It is clearly seen that the decomposed

granite can be compacted very easily. The field data show that the in-situ density is about 1.9 t/m³ for

decomposed granite, while the crushed mud stone has lower in-situ densities of about 1.70 t/m³. Thus

the densities for this test series were set at 1.9 t/m³ and 1.70 t/m³ respectively for the decomposed granite

and the crushed mud stone. Figures 21 and 22 show the liquefaction strength curves for these two materials. These figures indicate

that there is some effect of confining pressure on the available liquefaction strength. The effect is more

apparent for the crushed mud stone, and the decrease of strength due to the increase in confining pressure

is larger for this material. From the test results shown in the figures, it may be concluded that crushed

mud stone has higher liquefaction strengths than decomposed granite. However, it is not possible yet to

conclude that the difference in liquefaction strengths between the two materials is simply based on the

difference in the densities of the test specimens. For defining the liquefaction strength, it is necessary

to define the relative density of each soil and compare the liquefaction strengths at the same relative

density.

Figure 21. Liquefaction strength curve for decom

posed granite. Figure 22. Liquefaction strength curve for crushed mud stone.

There is a problem, however, to define the maximum and minimum density for such well graded

coarse fill materials. In Japan, the JGS standard for defining the maximum and minimum density is

applicable only for uniform sand. According to this procedure, the maximum density is obtained by

applying 100 blows to the side wall of a mold that contains the sand specimen. However, for the coarse

grained soil, this procedure is not applicable and some other method is required. On the other hand, the

procedure for obtaining the minimum density is applicable for a wider range of soils as it requires the

soil sample to be placed as gently as possible into a specific volume of container.

Iwasaki et al (1986) have proposed a procedure to obtain the maximum density for a well graded

coarse fill material, which involves the use of compaction test results. Figure 23 shows the results of

compaction tests with various compaction energies for both the decomposed granite and crushed mud

stone. The compaction density increases with the increase of energy, but it asymptotically approaches a

limiting value. This limiting value at high compaction

energy is defined as the maximum density for the fill material.

By applying the above procedure for defining the maximum density, the relative densities of the

decomposed granite and the crushed mud stone are determined for those specimen densities used in the

liquefaction test. The comparison of liquefaction strength in terms of relative density is made in Fig. 24,

which shows that the changes of liquefaction strength with relative density is very consistent using

Toyoura sand. It should be noted that the liquefaction strength is defined here as the stress ratio at 20

cycles of loading for $DA = 5.0\%$. The in-situ density of 1.9 t/m^3 for decomposed granite corresponded

to a relative density of 77%, and a similar procedure for defining the relative density should be used for

the liquefaction tests of well graded coarse fill materials.

4.2.2 Liquefaction strength: Effect of fines content

In Japan, the liquefaction potential of ground may be evaluated using various design codes, such as the

Road Bridges Design Specification. In these design codes, the use of liquefaction tests based on uniform

sand, i.e., Toyoura Sand, is extensively used. These design codes also provide formulas to account for

the effect of fines contents, the presence of which is generally assumed to enhance the liquefaction

strength.

After the earthquake, there were many liquefaction studies on the decomposed granite, and some

contradictory results on the effect of fines content on liquefaction strength have been reported. For

example, Sato et al (1996) examined the changes of

liquefaction strength with different levels of fines

content as shown in Fig. 25. In their tests, the fines content of the test specimens was varied by adding

the fines crushed from the original decomposed granite. The figure shows that the liquefaction strength 0.5 0.7 0.9 1.1 1.3 1.5 1.7 1.9 2.1 2.3 0 5 10 15 20 25 30 D r y D e n s i t y (t f / m

3) Compaction Energy (kgf/cm²) Crushed Mud Stones
Decomposed Granite (This study) Decomposed Granite
(Iwasaki et al.) Both cases, $p_{dmin} = 1.40tf/m^3$ assumed

Figure 23. Maximum density as obtained from

compaction testing. Figure 24. Comparison of liquefaction strength using relative density. Fines Content 0% Fines Content 10% Fines Content 20% Fines Content 40% 0.4 0.3 0.2 0.1 0.0 1 10 100 500 No. of Cyclic Loading C y c l i c S t r e s s R a t i o

Figure 25. Effect on fines content on liquefaction strength.

decreases with the increase of fines content, and a minimum strength is mobilized with fines content of

about 20%. In the present study, similar liquefaction tests were carried out using the decomposed granite and

the crushed mud stone. Since there was a considerable difference in the liquefaction damage between

Kobe Port Island and Rokko Island, it was thought that the difference of fines content between the two

materials may have resulted in different liquefaction resistances. In order to compare the liquefaction

strength of two materials having an identical amount of fines content, it was decided to perform additional

liquefaction tests for the two materials by adjusting their gradation curves (i.e., fines content also) to be

the same. The gradation curve adopted for additional test specimens is the average curve shown earlier

on Fig. 19. The comparison of liquefaction strengths for the two materials is shown in Fig. 26. The figure

shows that the liquefaction strengths of decomposed granite with the average gradation are signifi

cantly lower than those obtained using the original gradation. Therefore the increase in fines content has

reduced the liquefaction strength, which is contrary to what is given in the Japanese design codes. On

the other hand, the liquefaction strengths of crushed mud stone with the average gradation are lower than

those obtained using the original gradation. This means that the decrease in fines content has resulted

in the lower liquefaction strength, and this is in accordance with the usual Japanese design codes.

Figure 26. Changes of liquefaction strength curves due

to fines content changes. Figure 27. Variation in liquefaction strength curves due to clay content.

The increase in fines content seems to have two different effects. For non-cohesive fines such as

decomposed granite, the presence of fines tends to decrease the liquefaction strength. A similar finding

is presented by Sato et al. (1996), see Fig. 25. On the other hand, the presence of cohesive fines, such as

those obtained from crushed mud stone, tends to increase the liquefaction strength. Kuwano et al. (1996)

has shown that the increase of plasticity index or clay content has a clear increase on the liquefaction

strength, see Fig. 27. Hence, it may be concluded that the amount of fines does not directly control

the changes in the liquefaction strength. Instead, the mineral nature of the soil particles could play a

significant role in whether the amount of fines would increase or decrease the liquefaction strength of

that soil.

4.2.3 Post liquefaction settlement

One of the significant liquefaction features at the man-made islands was the post liquefaction settlement.

For example, large settlements in the range of 20 to 30 cm were observed at Port Island. There are

several methods for estimating the post liquefaction settlements of liquefied ground, among which is

one proposed by Ishihara and Yoshimine (1992), and one by Shamoto et al. (1996). In both methods,

the volume compression after liquefaction is related to the maximum shear strain attained during the

earthquake. The other key parameter which is compared against the maximum shear strain is the pre

liquefaction density state. In the former, one needs to precisely evaluate the relative densities and, in the

latter method, the void ratio difference from the minimum.

As noted previously for the liquefaction test series, post liquefaction settlement tests were performed

by draining the excess pore pressures after the above mentioned triaxial liquefaction tests. The results

of these tests are used herein to examine the currently available prediction methods for post liquefaction

settlement as described above.

To utilize the method of Ishihara and Yoshimine (1992), the relative density of in-situ fill materials

needs to be determined. The relative densities of test specimens for the decomposed granite and the

crushed mud stone have been estimated using the results of compaction tests described earlier. Thus

it is possible to use the method of Ishihara and Yoshimine (1992) for predicting the post liquefaction

settlement.

Figure 28 plots the post liquefaction settlements against the maximum shear strain observed from

the triaxial test series. The data points for both the decomposed granite and the crushed mud stone are

presented. The relative density for the decomposed granite ranged from 65 to 87%, while for the crushed

mud stone, it was between 72 to 88%. The observed post liquefaction settlement was about 2 to 4% for

the decomposed granite, and about 1.5 to 3% for the crushed mud stone. The prediction based on the

Figure 28. Post liquefaction volumetric strain vs. maximum shear strain.

Figure 29. Post liquefaction compression versus cyclic shear stress ratio.

Figure 30(a). Post liquefaction compression versus

cumulative shear strain (decomposed granite). Figure 30(b). Post liquefaction compression versus cumulative shear strain (crushed mud stone).

Ishihara and Yoshimine (1992) procedure would give about 1 to 2% of post liquefaction settlement for

the relative densities between 70 to 90%. The prediction by Ishihara and Yoshimine (1992) does give

reasonable settlements, but there is so much scatter in the data points shown on Fig. 28. One of the observations from the liquefaction triaxial tests is that both the decomposed granite and

the crushed mud stone exhibit a gradual increase in shear strain under the repeated, undrained loadings.

This is somewhat different from the observed deformation behaviour of clean sand, which exhibits a

very sudden increase in shear strain when liquefaction is reached. Figure 29 illustrates also this unique

deformation behaviour of coarse grained well graded fill materials. In Fig. 29, all data points from the

post liquefaction tests were plotted against the shear stress level applied for the various test specimens.

It shows that the increase in shear stress level results in smaller post liquefaction settlement, which

is in contradiction to the standard notion of relating the maximum shear strain to post liquefaction

settlement. On the other hand, as these fill materials exhibit a cumulative increase of shear strain with

increasing cycles of load application, the post liquefaction settlement of such materials may be related

to the cumulative shear strain sustained during the entire undrained loading. Based on this hypothesis,

Fig. 30 was produced, which correlates the post liquefaction settlement to the cumulative shear strain.

It is clearly seen that the correlation is very good for the decomposed granite, while the correlation for

the crushed mud stone is somewhat less favorable.

5 SUMMARY

In this paper, the static and dynamic geotechnical properties of weathered granite have been reviewed.

For the static geotechnical properties, extensive research studies have been carried out on the behaviour

of weathered granite in the undisturbed state. The physical and mechanical properties are greatly influ

enced by the level of weathering, and various classification systems are available for defining the level of

weathering. However, there seems to be no universally applicable classification system to cover various

weathered granites that are deposited under widely different climatic, topographic and geomorphological

conditions. The most promising parameters to identify the level of weathering would be the following

two parameters: void ratio and ignition loss.

In order to understand the mechanical behaviour of undisturbed weathered granite, the role of the

initial soil structure and the particle breakage or crushing during shear needs to be examined for their

influence on the deformation and strength properties. The undisturbed sample often yields less stiff

deformation behaviour as compared with that of the disturbed (compacted) sample, and the difference

in the initial soil structure and in the particle crushing during shear has to be considered in explaining

such behaviour.

For the dynamic geotechnical properties, namely liquefaction properties, of weathered granite,

research has progressed rapidly since the Great Hanshin-Awaji Earthquake of 1995. There is a need

to collect more data related to the liquefaction behaviour of coarse grained and well graded fill mate

rials in order to identify the difference in liquefaction behaviour between the uniform clean sand and

the coarse grained and well graded materials. For the liquefaction behaviour of weathered granite, it

is important to understand the role of gradation characteristics and particle breakage during undrained

repeated shear that could affect the undrained behaviour.

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Characterisation and Engineering Properties of Natural

Characterization and engineering properties of the Old
Alluvium in Puerto Rico

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ABSTRACT: This paper summarizes the mineralogy, microstructure, index and engineering properties

of a weathered Old Alluvium in Puerto Rico. The vertical profile includes more than 25 m of Pleistocene

Old Alluvium, ranging from highly weathered Upper Clay, through less weathered Middle Zone, to

unweathered Lower Sand layers. The weathering process has produced a combination of primary miner

als, secondary clay minerals and Fe-oxides, forming a complex microstructure. The intact soil comprises

a stiff network of interconnected silt-sized aggregates, made up of clusters of pellets formed from stacks

of clay particles. The pellets and aggregates are coated and cemented together by Fe-oxides forming

stable, cemented pseudo-silt or sand grains. A conceptual microstructural model is proposed to provide

a qualitative framework for explaining the relationship between macroscopic engineering properties and

degradation of microstructure. When the Fe-oxide coatings are broken down in 1-D consolidation tests,

there is a very large change in the compression, swelling and consolidation properties. These effects of

destructuring have important implications for ground engineering in the Old Alluvium.

1 INTRODUCTION

'Alluvium' is a general term for all transitory or permanent deposits of water-borne sediments (these

can include mixtures of gravels, sands, silts and clays) in stream channels, flood plains or deltaic

environments (Stokes & Varnes 1955, Edelman & van der Voorde 1963). The term 'Old Alluvium'

usually refers to Pleistocene-age alluvial deposits that are found in river terraces. Soil scientists consider

Old Alluvium as 'well-developed soils' which have generally been exposed to subaerial environments

(due to sea level change or tectonic uplift) and hence, have undergone significant physical and chemical

alterations. Geographical location and climate are key factors in determining the intensity of weathering.

Old Alluvium deposits are found widely in South East Asia (extinct drainage system of the South China

Sea; Gupta et al. 1987), in the major river basins of South Asia (Ganges, Brahmaputra and Indus),

sub-Saharan Africa (Niger, Congo), and South America (Orinoco), as well as Northern California. Old

Alluvium deposits in tropical regions are generally characterized by their low natural fertility, types of

clay minerals, low contents of carbonates and organic matter, and gley-mottling texture.

The Old Alluvium in Puerto Rico is distributed over a gently sloping coastal plain located in the

central-eastern part of the north coast of the island (Deere 1955, Kaye 1959, Pease & Monroe 1977). In

fact, it blankets much of the northeastern coastal plain, higher stream terraces, and part of the central

upland (Fig. 1). In addition, it underlies most of the present-day flood plain sediments of Holocene age

(e.g. recent littoral deposits and recent alluvium). The deposits are referred to locally as the Hato Ray

Formation because the deposit is exceedingly well exposed at the surface in the Hato Ray district of

San Juan.

The contact between the Old Alluvium and the underlying bedrock is generally sharp. It is commonly

believed that an eroded peneplain of Middle Tertiary limestone exists beneath the Old Alluvium in

San Juan (Fig. 2). As the coastal plain was developed through various stream channels, this deposit is

highly variable in thickness, but has a minimum depth of 25 m (Kaye 1959, Pease & Monroe 1977).

Figure 1. Old Alluvium in Puerto Rico.

The variation in thickness may also result from the complex eroded surface of the limestone bedrock

on which the Old Alluvium rests unconformably. The Authors have obtained block samples of the Old

Alluvium from a tunneling project. The Old Alluvium in San Juan, Puerto Rico was encountered during underground construction of the

Tren Urbano in Río Piedras. Four different tunneling techniques were used for this 1.5 km section of the

project: 1) conventional open cut and cover; 2) short sections of NATM tunnels; 3) twin tunnel bores

using an Earth Pressure Balance tunnel boring machine; and 4) a stacked drift cavern for the new Río

Piedras station (Whittle & Bernal 2003). The tunnel construction caused large surface settlements in

Figure 2. Soil profile of the Old Alluvium showing the sampling locations.

some areas. For example, in the stacked drift section, despite extensive compensation and permeation

grouting, movements exceeded 140-160 mm in some locations.

Block samples of the Old Alluvium were taken from the fresh cutting surfaces from the cut-and-cover

and stacked drift excavations as shown in Figure 2. The quality of all block samples was assessed by

transmission X-ray radiography, and only those blocks showing no cracks or other detectable disturbance

were used for consolidation and strength testing and for observing the intact structure. Geotechnical

site investigations, laboratory testing, and subsequent excavation and construction provided a great

opportunity to study extensively this soil in detail, including geological history, site characteristics,

mineralogy, soil structure, and index and engineering properties.

The vertical profile of the Old Alluvium in Río Piedras can be broadly sub-divided into three main

units (after WCC 1997):

- Upper Clay (UC), with an average thickness of 9 m, is medium stiff and brittle and consists mostly of

red (or mottled red and white) silty clays with complex patterns of white veins.

- Middle Zone (MZ), 10 m thick, is very stiff and brittle and consists of light brown to yellowish sandy

clay, silt, or clayey sand with occasionally inter-bedded sand lenses and pockets.

- Lower Sand (LS), is a layer of relatively clean sand or silty sand. Only bucket samples have been

obtained for this layer.

This classification is of great convenience for engineering practice. However, the actual boundaries

and transitions between these layers are much less distinct

in practice and vary with locations and depths.

As shown in Figure 2, the groundwater table is 22 m below ground surface in the Lower Sand layer.

2 ENGINEERING GEOLOGY

2.1 Parent material

The territory of Puerto Rico can be divided into three main geographic divisions (Fig. 1a): 1) a mountain

ous core that makes up the southern two-thirds of the island, also known as the central upland province;

2) a belt of rugged Karsts topography in the north-central and northwestern parts of the island; and 3) a

discontinuous fringe of relatively flat coastal plains distributed around the island. According to USDA

(1978), the Old Alluvium is derived from parent rocks in the central upland province which include:

- Intrusive igneous rocks, mainly granodiorite and quartz-diorite;
- Extrusive basic volcanic rocks, such as lava, tuff, breccia;
- Sedimentary rocks, such as limestone, tuffaceous sandstone and siltstone. The major rock-forming minerals contained in these rocks in the central upland were summarized in

a classic study by Kaye (1959) as follows:

- Sedimentary rocks: calcite, marly clay, quartz, kaolinite;
- Intrusive igneous rocks: albite, quartz, calcite, augite, chlorite;
- Volcanic rocks: feldspar, quartz, mica, pyroxene, albite, hornblende, chlorite.

2.2 Depositional environment

The Old Alluvium was formed through the accumulation of alluvial and colluvial sediments, in a series

of coalesced alluvial fans which are slightly dissected by small streams. The evidence of occasional

stratification, sand pockets and lenses, the lack of marine fossils, and the areal relationship with estab

lished fluvial deposits suggest very strongly that this formation was originally deposited as a piedmont

alluvial plain (Deere 1955). Numerous sea level fluctuations in the Pleistocene glacial epoch caused a

complex stream system with meandering stream channels to develop. During periods of glacial retreat,

sea level rose and finer-grained deposition generally occurred. In periods of glacial advance, sea level

fell as water was taken up in expanding glacial ice. As a result, stream gradients and energy increased and

caused down-cutting through and reworking of previously deposited coastal plain sediments. Therefore,

deposition occurred in a fresh water environment in stream channels and river terraces.

2.3 Post-depositional processes and timing

The alluvial origin of the Old Alluvium may be misleading to geotechnical engineers if no further

investigation is conducted. After initial deposition, the material became exposed to the atmosphere due

to either sea level changes or tectonic uplifts. In the sub-aerial environment, tropical weathering and

new soil-forming processes are initiated, causing continuous alteration of the composition, structure and

engineering properties of the soils. Initial alluvial deposition took place in the early Pleistocene (i.e. 1.0-1.6 million years ago). The

current material consists of thoroughly decomposed sands and gravels with extensive conversion of

unstable primary minerals (i.e. except quartz and orthoclase) into clays or other secondary minerals

(such as oxides). There is an almost complete lack of relationship with present-day stream alluviation

(Kaye 1959). Significant surface erosion may also have occurred in the tropical wet climate (with

abundant rainfall and frequent storms). Desiccation commonly takes place following tectonic uplift or lowering of the groundwater table and

is especially common in climates where there is a high rate of evaporation during dry seasons. The upper

units of the Old Alluvium (UC, Fig. 2) appear to have undergone intensive desiccation, resulting in a

well developed cross-patterned network of shrinkage cracks. Moreover, it appears that desiccation may

have also caused the formation of clay nodules in the shallow surface layer. It is very difficult to quantify the stress history of a complex formation such as the Old Alluvium.

However, a review of the geological processes affecting its formation suggests that the current formation

is overconsolidated:

- Transport and deposition: deposition (loading process);
- Sedimentation, compression and consolidation (loading);
- Uplift (causing exposure to the atmosphere) and desiccation (loading);
- Weathering, which usually decreases the density of the deposit (unloading);
- Erosion (unloading).

3 SOIL COMPOSITION

The complete determination of soil composition requires positive identification and quantification of

the solid minerals and the measurement of the pore fluid chemistry.

Table 1. Mineralogy of the Old Alluvium. Fraction in UC
Fraction in M2

Mineral Ideal Formula (%) (%)

Quartz α -SiO₂ 22 32

Orthoclase KAlSi₃O₈ 9 17

Kaolinite Al₂Si₂O₅(OH)₄ 34 23

Nontronite Ca_{0.25}Si_{3.5}Al_{0.5}Fe₂O₁₀(OH)₂ 20 13

Montmorillonite Ca_{0.25}Si₄Al_{1.5}Mg_{0.5}O₁₀(OH)₂ 0 2

Hematite α -Fe₂O₃ 3 0

Goethite α -FeOOH 6 3

Total 94 90

Clay fraction 63 41

3.1 Solid minerals

The mineralogy of the deposit was analyzed from homogenized bucket samples from the UC and M2

zone using a variety of qualitative and quantitative techniques including X-ray diffraction (XRD), X-ray

fluorescence (XRF), cation exchange capacity (CEC) measurements, thermal analysis, and selective

chemical dissolution of Fe-oxides (Zhang 2002).

To facilitate the identification of clay minerals, the clay fraction (<2 μ m) was separated using sed

imentation and centrifugation. Two sample preparation methods were used to positively identify clay

minerals by XRD: oriented aggregates and random powder (Zhang 2002). The existence of clay minerals

was further verified by XRF and thermal analyses. The minerals in the coarse fraction (>2 μ m) were

mainly identified by random powder XRD and differential thermal analysis (DTA). Table 1 summarizes

the fractions of the major minerals existing in the UC and M2 layers by weight.

There are two major coarse-grained minerals present in the Old Alluvium: quartz and orthoclase (K

feldspar). These two minerals exist at greater fractions in the M2. It is quite reasonable to find these two

minerals in the weathered Old Alluvium, as they are the two most weathering resistant primary minerals

(Dixon & Weed 1989), and intense tropical weathering has converted all other less stable ones into clays

or oxides.

The two major clay minerals are kaolinite and smectite, which are present in both layers. The clay

minerals were principally identified by XRD patterns obtained on oriented aggregates with various

treatments. Figure 3 shows results for the UC and M2 specimens saturated with K⁺ and Mg²⁺ cations at

a range of temperatures and the effects of glycerol solvation. The smectite in the M2 includes two species,

nontronite and montmorillonite, while the latter is not present in the UC. Two Fe-oxides - hematite (α

Fe₂O₃) and goethite (α -FeOOH) are present in the UC at 3% and 6% respectively, while only goethite

is present in the M2. As the particle size of Fe-oxides is usually less than 0.1 μ m, they belong to clay

fraction. Therefore, the total clay fractions for the UC and M2 are 63% and 41%, respectively.

The phases and fractions of these minerals clearly reflect that the UC is more weathered than the M2.

For example, quartz and orthoclase have a greater fraction at depth, indicating that the conversion of

primary minerals into secondary ones is not as advanced in the M2. The existence of smectite indicates

that the Old Alluvium is not as well developed as other tropical residual soils that contain only kaolinite

as the major clay mineral (Fookes 1997). Since this deposit was formed in the early Pleistocene, the time

allowed for tropical weathering may not be sufficient for complete conversion of smectite into kaolinite.

Therefore, the mineralogy of this Old Alluvium agrees well with its geological history and formation

process.

3.2 Pore water and chemistry

To achieve further understanding of the effects of weathering on soil behavior, efforts have also been made

to measure the chemical properties of the Old Alluvium, as summarized in Table 2. The NaF pH of both

UC and MZ layers is less than 9.4, indicating that there is no detectable amount of amorphous minerals

present in the Old Alluvium (SSL 1996). The other three pH values reflect the chemical properties (e.g.

soil acidity), type of inorganic and organic matters, amount and type of exchangeable cations and anions,

salt or electrolyte content, and CO₂ content. The results agree well with the usual pH sequence of most

soils, i.e. 1:1 water pH > 1:2 CaCl₂ pH > 1:1 KCl pH. However, the MZ is always less acid than the UC,

as indicated by the pH values. KCl pH is an index of soil acidity. As the KCl pH is less than 5, significant (a) UC (b) MZ 0 5 10 15 20 25 30 20 [degree] I n t e n s i t y Mg 2+ , RT Mg 2+ +Glycerol, RT K + , 300 o C S (0 0 1) d = 1 4 . 6 2 K (0 0 1) d = 7 . 2 4 K (0 0 2) d = 3 . 5 6 K (0 0 2) d = 3 . 5 8 S (0 0 1) d = 9 . 7 9 G , d = 4 . 1 7 Q , d = 3 . 3 3 S (0 0 1) d = 1 8 . 0 9 K (0 0 1) d = 7 . 1 7 S (0 0 6) , d = 2 . 9 7 S (0 0 3) d = 5 . 8 9 S (0 0 4) d = 4 . 4 5 S (0 0 3) , d = 3 . 3 2 S (0 0 1) d = 1 1 . 3 0 K (0 0 1) d = 7 . 2 1 K (0 0 2) d = 3 . 5 5 Q , d = 3 . 3 3 S (0 0 2) d = 4 . 8 6 Q , d = 3 . 3 2 K + , 500 o C Q , d = 3 . 3 4 I (0 0 1) d = 9 . 8 9 I (0 0 2

) d = 4 . 9 8 K (0 0 2) d = 3 . 5 6 S (0 0 3) / I (0 0 2) d = 4 . 9 8 I (0 0 1) d = 1 0 . 0 6 P (0 0 3) d = 3 . 1 0 I (0 0 1) , d = 9 . 9 4 P (0 0 1) , d = 9 . 1 9 S (0 0 2) , d = 8 . 9 9 0 5 10 15 20 25 30 20 [degree] I n t e n s i t y Mg 2+ , RT Mg 2+ +Glycerol, RT K + , 300 o C S (0 0 1) d = 1 4 . 4 7 K (0 0 1) d = 7 . 1 5 K (0 0 2) d = 3 . 5 4 K (0 0 2) d = 3 . 5 4 S (0 0 3) d = 4 . 9 3 H / G d = 2 . 6 8 H d = 2 . 6 9 H / G d = 2 . 6 9 G , d = 4 . 1 4 Q , d = 3 . 3 3 S (0 0 1) d = 1 8 . 0 9 S (0 0 2) d = 8 . 9 3 K (0 0 1) d = 7 . 2 8 S (0 0 6) d = 3 . 0 0 S (0 0 3) , d = 5 . 9 6 Q , d = 3 . 3 4 G , d = 4 . 1 8 G , d = 4 . 9 8 S (0 0 4) , d = 4 . 4 7 S (0 0 2) d = 4 . 8 4 S (0 0 1) d = 1 1 . 6 8 K (0 0 1) d = 7 . 2 2 K (0 0 2) d = 3 . 5 5 K (0 0 2) d = 3 . 5 1 Q , d = 3 . 3 4 S (0 0 1) d = 9 . 6 9 K (0 0 1) d = 7 . 2 1 H / G d = 2 . 6 9 K + , 500 o C

Figure 3. XRD patterns of oriented aggregates obtained from the <2 μm fractions.

Note: d = layer spacing, in Å; Mg 2+ and K + = clays are saturated with exchangeable cations Mg 2+ and K + ;

RT = room temperature; S = smectite, K = kaolinite; G = goethite; H = hematite; Q = quartz; I = illite.

amounts of Al 3+ are present in the solution or as exchangeable cations. Carbonate is not present in the

deposit, as it is not stable in the wet tropical climate. The high precipitation ensures that all soluble

salts are removed quickly by percolating water and hence, there is a very low content of these salts in

either the UC or M2. Although vegetation is abundant there is also a very high rate of decomposition

of organic substances (high microbial activity) in the tropical climate and again, the organic content is

very low. Table 2. Some chemical properties of the Old Alluvium. Properties UC M2 NaF pH 7.67 7.53 1:1 water pH 4.90 5.38 1:2 CaCl 2 pH 4.03 4.48 1:1 KCl pH 3.48 3.84 Soluble salt (ppt) 0.086 0.048 Carbonate (%) 0.00 0.00 Organic matter (%) 0.10 0.00

4 STRUCTURE

In order to understand the engineering properties of

natural soils it is important to take into account the effects of structure. Soil structure refers to both fabric, including particle association and arrangement, and interparticle forces that are not purely frictional in nature (Lambe & Whitman 1969). Due to its particulate nature, a soil's fabric occurs at both particle level (i.e. microfabric) and soil assemblage level (i.e. macrofabric). Since the Old Alluvium was initially formed by alluvial deposition and then underwent tropical weathering, the structure of the current soil reflects the influences of these geological processes. Cementation is another important component of structure, which is usually formed through the process of reprecipitation. This form of interparticle force is associated with physical links caused by precipitation of minerals between particle contacts (intergrowth). As a result, cementation usually bonds particles together and can cause the formation of relatively large, stiff units referred to as 'aggregates'.

4.1 Macrobatic

Here 'macrofabric' refers to soil structure at the macroscopic level (i.e. those features that can be observed visually). The macrofabric was mainly obtained by observing the in-situ soil profile exposed by excavation of large intact block samples. It is clear that the current macrofabric of this deposit is formed through initial deposition, subsequent tropical weathering, and the effects of environment (e.g. remnant vegetation roots, dissolution of the limestone bedrock, high evaporation rate in the hot climate, etc.). Typical features of macrofabric observed in this deposit include stratification, mottling and coloration, and white venation.

4.1.1 Stratification

Stratification is a typical feature of most alluvial deposits. As the Old Alluvium was initially formed through a series of coalesced alluvial fans which were dissected by small streams, the irregular stratification distributes almost everywhere within the deposit. Occasionally, sand pockets or lenses can be observed through exposures on slopes or cuttings (Kaye 1959). However, due to the long-term alteration caused by tropical weathering, stratification tends to diminish and is not apparent in places where intensive weathering has converted the original sand and gravel grains into clays. Therefore, weathering tends to make this deposit more homogeneous at the large scale.

4.1.2 Mottling and coloration

Field observation and laboratory identification have found that the most striking appearance of this soil is the reticulated mottling and coloration (Fig. 4). For example, the UC layer consists mostly of red, or mottled red and white, silty clays, with a somewhat crisscross pattern of white and buff clay veins. Mottling usually occurs at shallow depths. The mechanism causing white mottling is the same as that of white venation, which will be described later. The color of this material changes with depth. A general trend over depth is that the soil color changes from bright red to brown in the UC, to yellowish brown or yellowish gray in the M2. These color changes are caused by the variations of mineral phases and amounts of Fe-oxides with depth. As shown in Table 1, hematite and goethite are present in this deposit. The former

usually has a bright red color, while the latter light brown to yellowish color (Schwertmann & Taylor

1989). Due to the higher pigmenting power of hematite, the presence of goethite in the UC is partially

masked by hematite. The brown to yellowish color of the MZ reflects the presence of goethite and the

absence of hematite. These observations agree well with the results of mineralogical analysis (Table 1).

Figure 4. Coloration and macro-fabric of intact block samples of the Old Alluvium. Table 3. Chemical composition of the bulk and white vein regions of the UC. Fraction in bulk Fraction in white vein Major element (%) (%) Na 0.10 0.07 Mg 0.87 0.57 Al 8.54 13.66 Si 22.34 27.01 K 1.15 1.10 Ca 0.16 0.20 Ti 0.50 0.67 Fe 9.24 3.70 Note: the fractions are weight percentage based on air-dried samples.

4.1.3 White venation

Another important feature of the macrofabric is the white venation through both the UC and MZ (Figs. 4b,

c), which makes the material heterogeneous in a relatively small scale. This phenomenon is also called

'gleying' by geologists (Kaye 1959). These veins have a crisscross pattern, an inch or less in width.

Most veins are more or less circular in cross-section, while some may extend over a relatively large

plane. The central part of a typical vein consists of the white clay, with the brown or buff clay forming

an intermediate zone separating the white clay from the red soil matrix. Chemical analysis shows that

the white vein in the UC has a higher content of Si and Al and much lower content of Fe than the bulk

UC sample (Table 3). As suggested by Kaye (1959), this white venation is introduced into this deposit

Figure 5. Pictures showing white veins in the UC caused by localized weathering along root paths.

through localized weathering along previous cracks or live

root paths, during which Fe-oxides (including red hematite and brown goethite) are further dissolved by organic acids released from dead plants or live roots.

There are three distinct categories of white veins with different origins:

- Shrinkage cracks, which have a complex reticulated pattern and only occur in the shallow UC layer

(Fig. 4a). This is a type of localized weathering along shrinkage cracks by organic acid released from

dead plants. During the dry periods in a tropical climate, the high evaporation usually causes strong

desiccation (drying) and hence shrinkage cracks appear near the ground surface. After cracking,

subsequent rainfall brings water flow along these cracks and the soil swells to fill them. In the

meanwhile, dissolution of Fe-oxides along these cracks takes place due to percolating surface water

charged with organic acids released from the decomposed dead plants, resulting in white veins in the

red soil matrix. In fact, mottling is also caused by such mechanism in the shallow depth.

- Collapsing cracks, which have a relatively large planar area occur in both the UC and MZ (Fig. 4c).

They are formed through the same mechanism as those along shrinkage cracks, except that these

veins have a different origin of cracking. Three possible geological processes may introduce cracks

of this type: i) faulting and collapsing caused by the dissolution of the underlying limestone (Fig. 2);

ii) tectonic movements (Kaye 1959); or iii) rebounding by weathering and erosion, since weathering

tends to decrease the density of the initial material.

- Root paths, which usually have a circular cross-section and produce an uncommon reticulation pattern

(Fig. 5). To gain nutrients, live roots release organic acids or chelates that attack and dissolve Fe-oxides,

leaving white clays in place. This kind of localized weathering has been observed by soil scientists in

reddish soils (Schwertmann & Taylor 1989).

In summary, both live roots and dead plants promote the formation of white veins. Despite of their

different origin, these white veins contain little or no Fe-oxides, and hence their engineering properties

may be different from that of the surrounding red soil matrix. For example, the white clay is more plastic

than the red and contains a greater fine fraction (Kaye 1959). In addition, white veins along previous

cracks definitely have a lower shear resistance than the intact red soil matrix. Therefore, white veins

give the soil not only non-uniformity in appearance, but also small-scale heterogeneity in engineering

properties.

4.2 Microstructure

The microstructure of the Old Alluvium was characterized by both direct observations and indirect

examinations. The former includes Environmental Scanning Electron Microscopy (ESEM), while the

latter consists mainly of slaking, Cation Exchange Capacity (CEC), selective chemical dissolution, and

particle size analyses.

4.2.1 Direct observation of microfabric

A clear understanding of the microstructure requires direct observation of the physical arrangement of

particles which can be achieved by scanning electron microscopy (SEM). SEM has been widely used

to observe the microstructure and micromorphology (e.g. particle size, shape and boundary) of soils

(McHardy & Birnie 1987, Mitchell 1993). Conventional SEM requires that the pore fluid is removed and

special sample preparation techniques (e.g. freeze-drying, critical-point drying) are needed to minimize

sample disturbance. However, the newly developed environmental SEM (ESEM) can examine samples

in wet mode, and hence, little sample preparation is required. This is a great advantage in minimizing

disturbance to the intact structure. Microscopic observation of the intact fabric of the Old Alluvium, including particle size and shape,

particle association and arrangement, and pore spaces, were carried out using an FEI/Philips XL30 FEG

ESEM equipped with an energy dispersive X-ray spectrometer (EDXS). To observe the intact structure,

high-quality undisturbed samples were used, and the surfaces chosen for study should reflect the original

fabric of the soil. Therefore, the fresh surface of a small test specimen was obtained by breaking an intact

soil block (i.e. fracturing instead of cutting), and then using an adhesive tape to peel off any loose grains

from the newly exposed surface.

4.2.1.1 Particles and their Association

With the high resolution (0.1-1.0 μm) provided by ESEM, clay sized particles become visible, and it is

possible to observe the size and shape of individual clay particles. However, careful examination of the

intact soil under ESEM found neither distinct boundaries between two 'grains' nor clay particles with a

clearly defined shapes and edges (Fig. 6). Here the term 'grain' refers to a unit with a clearly defined

physical boundary, even though it may be composed of smaller individual particles. As mentioned earlier,

mineralogical analysis found kaolinite and smectite in the deposit (Table 2). Usually, kaolinites are flat

and thick platy-shaped particles, whereas individual smectite particles are delicately thin and curved

flakes (Borchardt 1989). Smectite may also exist as 'domains' wherein several nearly parallel 2:1 layers

are bonded by high valence interlayer cations (e.g. Ca^{2+} , Mg^{2+} , Fe^{3+}) forming thicker units referred to

as 'quasi-crystals' (Güven 1992). However, individual clay particles were not discernible in the intact Old Alluvium under ESEM.

Instead, all clay particles seemed to be stacked together into 'pellets' of 10-20 μm . Here a 'pellet' is a

kind of elementary particle arrangement where a group of clay particles or domains are associated in a

mainly parallel configuration (i.e. mostly face-to-face orientation). Gens and Alonso (1992) discussed

the wide occurrence of such elementary particle arrangements in natural, compacted, and resedimented

soils. For the Old Alluvium, the reasons why such clay pellets are formed may include: 1) the clays

are secondary minerals formed through weathering, during which the precipitation or recrystallization

process tends to form a group of clay particles associated with face-to-face orientations; and 2) the pore

fluid has a very low salt concentration (Table 2), which inhibits flocculation (i.e. edge-to-face orientation)

of clay particles. In fact, the formation of similar clay pellets has been observed in reservoir sandstone

pores and natural soils (Wilson 1987, Dixon & Weed 1989).

Further observations of the ESEM micrographs revealed that the clay pellets appeared to be covered

by a thin film or coating (Fig. 6). The outer surfaces of the pellets are coated (both the basal planes

and edges) leaving no clear shape or boundary of individual clay particles. Moreover, the pellets are

inter-connected by bridges. This is demonstrated in Figs. 6a, b. As described later, the presence of

cementation (the coating and connections) in the UC and MZ was further confirmed by observing the

micromorphology of samples treated by chemicals to dissolve the cementation. The above observations focused on the scale less than 10 μm . If the magnification is decreased

to the scale of 50-100 μm , another feature of microstructure becomes evident. This is the existence

of aggregates, which consist of groups of inter-connected clay pellets. As shown in Figure 7, these

aggregates are approximately spherical or semi-spherical in shape with a characteristic size of 50-

100 μm with a bouldery surface texture resembling a 'cauliflower'. Further observation found that the

aggregates are also bridged together to form an interconnected network of aggregates (i.e. they do not

exist as individual separate 'grains'). Due to the coating on the surfaces of the clay pellets, the connections

between clay pellets and between aggregates, the intact material behaves as a stiff, cemented granular

medium.

4.2.1.2 Pore spaces

The micrographs also revealed certain features of the pore spaces, including pore size, shape, intercon

nection and distribution. As shown in Figure 7b, there are two different types of voids: inter-aggregate and

Figure 6. ESEM micrographs obtained on intact samples: (a) the UC layer showing coating on surfaces of grains

and connections between grain contacts; (b) the M2 layer showing a thin layer of coating and bridge connection

between two grains; (c) the M2 layer showing a connected network of grains; and (d) the UC layer showing that the

edge (where the arrows point) of a grain is coated and no individual clay particles are discernable.

Figure 7. ESEM micrographs obtained from the intact UC showing (a) aggregates and (b) pore spaces.

intra-aggregate. That is, the soil possesses a dual porosity, which is generally observed in many natural

soils with aggregate formations (Gens & Alonso 1992, Mitchell 1993, Righi & Meunier 1995). For the

Old Alluvium, the inter-aggregate pore diameter is roughly 50-80 μm , while intra-aggregate pores are

smaller than 10 μm . Clearly, most inter-aggregate pores are connected to each other (Fig. 7b), resulting

in a high hydraulic conductivity for the intact soil ($\sim 0.5\text{--}1.0 \times 10^{-6} \text{ cm/s}$). Since ESEM can examine

the sample with a large depth of field (McHardy & Birnie 1987), the pore shape in three dimensions can

be roughly estimated. Most of the inter-aggregate pores have irregular shapes and are connected to each

other by channels in different directions. However, it appears that there is lack of good interconnectivity

among those intra-aggregate pores. This feature and the much smaller size of intra-aggregate pores make

them unavailable or difficult for water flow, and thus the porosity derived from intra-aggregate pores

contributes little to the hydraulic conductivity.

4.2.2 Identification of cementing agents

The ESEM observations show that the clay pellets are

cemented together to form aggregates. This

cementation is formed through precipitation or recrystallization between the contacts of clay pellets. The

precipitation of these secondary minerals may also take place on particle surfaces, resulting in a coating

covering clay pellets. In fact, the formation of coatings over sand particles has been widely observed

in sandstones (Pettijohn et al. 1972). Although it is clear that the intact structure of the Old Alluvium

possesses cementation bonding, separate studies have been needed to identify the actual cementing. It is not always easy to identify cementing agents in soils. A complete identification usually requires a

combination of several techniques, including such direct methods as X-ray diffraction (XRD) or EDXS

(under SEM/ESEM), and indirect methods such as selective chemical dissolution (SCD) combined with

particle size analysis. XRD can help to identify crystalline minerals, but is not effective in detecting

amorphous minerals (e.g. organic matter, amorphous silica). Another disadvantage of using XRD to

identify cementing agents is that, even if a mineral (e.g. silica) is identified by XRD, it is not clear

whether that mineral exists as cementation or occurs as particles within the aggregate. On the other

hand, since SEM/ESEM can observe the physical presence of cementation, EDXS equipped in ESEM

can analyze the chemical elements of the cementation. However, the actual crystal structure (i.e. what

mineral is formed by these elements) is still unknown. Therefore, other indirect methods are also required,

such as SCD combined with particle size analyses. The basic principle of SCD is to treat the soil with

specific chemicals that selectively dissolve and remove

only one kind of possible cementing agent.

Subsequent particle size analyses can measure the change in particle size characteristics caused by SCD.

Comparison of particle size distributions between samples with and without SCD treatment gives positive

information on the presence of cementation, due to the fact that soils with inter-particle cementation do

not readily disperse.

4.2.2.1 Selective chemical dissolution and particle size analyses

The common cementing agents occurring in natural soils include carbonates, organic matter, silica and

oxides (including hydroxides and oxyhydroxides) (Mitchell 1993). One or more of these substances

could be present in the Old Alluvium as cementing agents. The following SCD techniques were used

to investigate these possible cementing agents: The equivalent amount of calcium carbonate (CaCO_3)

was determined by treating the air-dried samples ground to pass a #325 ($<44 \mu\text{m}$) mesh with 1.0 N

hydrochloric acid (HCl) solution (USDA 1996, ASTM 1997). For complete dissolution of all Fe-oxides

(e.g. hematite, goethite), natural samples were treated 4-5 times with 1.0 g sodium dithionite ($\text{Na}_2\text{S}_2\text{O}_4$)

in a 70-80 °C 0.3 M sodium citrate ($\text{Na}_3\text{C}_6\text{H}_5\text{O}_7 \cdot 2\text{H}_2\text{O}$) solution buffered with a 1.0 N sodium bicarbonate

(NaHCO_3) solution (DCB, sodium dithionite-citrate-bicarbonate) (Schwertmann & Taylor 1989, Smith

1994, USDA 1996). Silica and organic matter were dissolved by treating natural samples with 0.1 N

sodium hydroxide (NaOH) solution and 30% hydrogen peroxide (H_2O_2) solution, respectively (USDA

1996). To quantitatively measure the disintegration caused by SCD, particle size analyses were performed

on untreated and treated samples by following the procedures of ASTM D422 (ASTM 1997). Note that

sodium hexametaphosphate ($\text{NaPO}_3)_6$ was used as a dispersant for all particle size analyses and that all

samples were subjected to one minute of stirring in a high-speed mechanical stirrer. The results of these analyses performed on the UC sample are shown in Table 4. As reported previously

(Table 2), there are no detectable carbonates in the deposit. Therefore, no particle size analysis was

performed on the sample treated with HCl solution. Table 4 shows little difference in particle size

distributions among the untreated samples and those treated with H_2O_2 , or NaOH (38% clay fraction).

However, the clay fraction of the sample treated with DCB increases significantly, showing that the

removal of Fe-oxides causes disintegration of the soil. Therefore, it can be concluded that the cementing

agents are Fe-oxides. It is also interesting to note the changes of the silt (2-75 μm) and sand ($\geq 75 \mu\text{m}$) fractions caused

by various SCD treatments. While the sand fraction of these samples remains unchanged, the increment

of the clay fraction of the sample treated with DCB comes from the reduction of the silt fraction. This

Table 4. Effect of various SCD on particle size distribution of the UC bucket sample. Clay fraction Silt fraction Sand fraction

Treatment	Mineral dissolved (%)	(%)	(%)
-----------	-----------------------	-----	-----

None	None	38.1	49.0	12.9
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H_2O_2	Organic matter	37.5	50.5	12.0
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NaOH	Silica	38.7	48.7	12.6
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DCB Fe-oxides 54.8 34.8 10.4 (a) (b)

0.0 1.0 2.0 3.0 4.0 5.0 6.0 7.0 X-ray energy [keV]

I n

t e

n s

i t y O Fe Al Si Fe K Mg Electron energy: 25 kV Tilt angle:
15 ° Take-off angle: 47.7° Time: 150 sec 0.0 1.0 2.0 3.0
4.0 5.0 6.0 7.0 X-ray energy [keV] I n t e n s i t y O K Al
FeK Si Fe Electron energy: 25 kV Tilt angle: 15° Take-off
angle: 47.7° Time: 151 sec

Figure 8. Spectra of EDXS analyses performed on the surface
of (a) a clay pellet and (b) a sand particle (probably

feldspar) in the UC sample.

fact, plus the very high initial hydraulic conductivity
($\sim 1.0 \times 10^{-6}$ cm/s) of the intact UC, indicates that

nearly all aggregates lie within the silt size range, which
is consistent with the ESEM observations of

the very large inter-aggregate pores described earlier.

4.2.2.2 EDXS analysis

With the high magnification provided by ESEM, qualitative
elemental analysis by EDXS could be

performed directly on selected areas. To further identify
the cementing agents, EDXS analysis was

performed on the surfaces of and the bridge connections
between clay pellets. Figure 8 shows the

typical results of EDXS analysis. It appears that Fe exists
on the surfaces of both clay pellets and

sand/silt particles. Since the electron beam in ESEM can
penetrate into the sample to a depth of several

micrometers (McHardy & Birnie 1987) and the size of
Fe-oxides is usually 0.05–0.1 μm (according to

Schwertmann & Taylor 1989), the chemical elements contained in the clay or sand-silt particles as well

as the elements in Fe-oxides are all shown in the recorded X-ray spectra. In fact, Fe-oxides coatings on

sand particles have been observed in sandstones (Pettijohn et al. 1972). Further analysis on the bridge

connections between clay pellets also found the presence of Fe. Therefore, it is appropriate to conclude

that Fe-oxides are the only minerals acting as cementing agents in this deposit. That is, Fe-oxides act as a

coating over clay pellets and silt-sand particles, and as bridge connections between pellets and between

aggregates. This can be further verified by the indirect methods discussed below.

4.2.3 Indirect determination of microstructure

The structure of the OldAlluvium was further determined by two indirect methods, namely measurements

of Cation Exchange Capacity (CEC) and slaking. Due to their negative surface charge, clay particles

attract cations which are exchangeable with other cations in solution. However, the presence of a coating

on clay surfaces can reduce the measured cation exchange capacity of clay minerals. Therefore, a

comparison of CEC measured on samples before and after the removal of the coating can provide further

proof of the presence of cementation. On the other hand, slaking is a basic test for distinguishing between

soil and rock (Morgenstern & Eigenbrod 1974, Mitchell 1993). Upon unconfined immersion in water, an

initially intact piece of cohesive soil may disintegrate into a pile of pieces or sediment of small particles.

However, a soil with cementation in its natural intact state (i.e. without drying or destructuring) will not

slake if the cementation is strong enough to sustain its self-weight and other forces induced by wetting.

Therefore, slaking is another indirect way to identify certain structural features.

4.2.3.1 Cation exchange capacity

Determination of CEC was performed on natural samples and on samples treated with DCB to remove

Fe-oxides. The procedures of Hendershot and Duquette (1986) were followed here. A sample was first

washed four times with 0.5 N BaCl₂ solution (pH = 8.0), then shaken in a mechanical wrist-action

shaker to accelerate cation exchange, washed again twice with distilled water, and finally washed twice

with 0.0007 N BaCl₂ solution. The difference in Ba concentration in samples before and after cation

exchanging can be used to calculate the CEC of the soil. Of the minerals present in the Old Alluvium,

only the kaolinite and smectite have the capability to exchange cations with solution, while the Fe-oxides

have zero CEC (USDA 1996). Table 5 shows the results of the CEC measurements. Clearly, the samples

treated with DCB have much greater CEC than the natural samples without treatment. Furthermore,

even though the UC layer has a higher clay mineral fraction than the MZ, the CEC of the natural UC

is smaller than that of the natural MZ, which is not possible if all clay particles were dispersed. This is

attributed to the higher Fe-oxides content in the natural UC, which provides more coatings of the clay

particles.

4.2.3.2 Slaking

Slake tests were performed to examine the stability of the intact UC sample in three different fluids: i)

distilled water; ii) glycerol solution; and iii) DCB solution. The glycerol solution has a glycerol/water

volume ratio of 1 : 7 and is generally used to solvate and expand smectite for XRD analysis (Novich &

Martin 1983, USDA 1996). After glycerol solvation, the interlayer spacing of fully hydrated smectite can

expand from ~1.2-1.4 nm to ~1.8 nm and a soil containing smectite should exhibit significant swelling.

The DCB solution consists of 80 ml 0.3 M sodium citrate solution, 10 ml 1.0 M sodium bicarbonate

solution, and 2.0 g sodium dithionite (USDA 1996). After immersion in these fluids, the slaking process

of an intact piece of soil was observed and recorded over time. Table 6 summarizes the observed slaking reaction of the intact UC samples. Although the natural UC

sample is unsaturated and contains a significant amount of smectite, the intact soil remained stable in

both water and glycerol solution for days. This implies that the lack of swelling is caused by the Fe-oxides

coating preventing the molecules of water or glycerol from direct interaction with smectite. Hence, no

double layer expansion occurs on the external surfaces of smectite particles and no swelling is caused

by the formation of cation-glycerol complex in the interlayer spaces of smectite. The slaking phenomena observed in the DCB solution are totally different. After immersion, the intact

soil piece slowly changed its color from red or brown to gray, indicating that Fe-oxides were gradually

dissolved. This was accompanied by a change of the solution's color to yellow. However, the solution was

still clear and transparent, indicating that no fine particles were dispersed into the solution. After two

Table 5. CEC of natural and DCB-treated samples. CEC

Sample (meq/100 g clay) Note

Natural UC 29.1 Disturbed UC with Fe-oxides cementation

DCB-treated UC 52.1 Without Fe-oxides cementation

Natural MZ 35.6 Disturbed MZ with Fe-oxides cementation

DCB-treated MZ 45.3 Without Fe-oxides cementation

Table 6. Observed slaking reactions of the intact UC block sample in three different fluids.

Fluid Reaction of the sample Final sample condition Change of the fluid

Water No reaction Same as the initial intact No color change

Glycerol No reaction Same as the initial intact No color change

solution

DCB solution Very slow cracking, but no Red color disappeared, the sample The solution became swelling and no dispersion of became gray and was fractured yellow, but was still fine particles into several smaller pieces transparent

First DCB, Quick swelling and selfA pile of dispersed fine The solution became

then glycerol peeling, dispersion of fine particles with an intact inner cloudy

solution particles into the solution core

days, the initially intact soil piece cracked and split into several smaller parts, but with only slight volume

expansion. Finally, a sample was first immersed in DCB for two days and then immersed in glycerol

solution. The most striking reaction was the significant swelling and quick self-peeling. Moreover, fine

clay particles became dispersed within the solution, which made it cloudy. After one day, a stable and

intact inner core was left with an exposed fresh red surface, indicating that the DCB solution only

dissolved the Fe-oxides in the outer shell of the soil sample, but left the inner core unattacked. This is

probably due to the fact that the amount of chemical reagents in the DCB solution could only dissolve

the Fe-oxides in the outer shell. The reason why the outer shell of the soil with dissolved Fe-oxides did

not disperse and swell significantly is probably because the salt concentration in the DCB solution was

sufficiently high to depress the double layer and hence greatly reduce the swelling of smectite.

In summary, from the slaking response in water and glycerol solution, it can be inferred that these

molecules could not reach the negatively charged outer surfaces or the interlayer spaces of the smectite in

order to cause either an expansion of the double layer or further hydration of the interlayer cations. This

indicates that the smectite particles in the intact sample are covered by a coating. Slaking in the DCB

solution dissolves Fe-oxides and subsequent immersion in the glycerol solution causes rapid swelling

and disintegration.

4.2.3.3 ESEM observation of the DCB-treated samples

The ESEM micrographs of the intact samples indicated that a coating masks the micromorphology of

individual clay particles and that cementing bridge connections exist between both the clay pellets and the

aggregates. Further ESEM micrographs have been obtained on samples pre-treated with DCB to study

the change in micromorphology of the clay particles caused by the removal of Fe-oxides. After the DCB

treatment, samples were washed several times with distilled

water to remove the soluble salts through

centrifugation, and the wet paste directly examined under ESEM. Since samples were free of Fe-oxides

and became completely dispersed, ESEM observation on the DCB-treated samples could focus only

on the micromorphology of the individual clay particles. Figure 9 shows that individual clay particles

become visible with a clear shape and boundary. For instance, kaolinite exists as thick, flat plates with

an irregular shape, while smectite is present as delicately thin and curved flakes. As expected, no coating

or connections were detected in these dispersed samples.

4.2.4 Summary of microstructure

The microstructure of the Old Alluvium has been characterized through a combination of direct micro

scopic observation by ESEM and indirect examinations by slaking, CEC measurement, EDXS, SCD

and particle size analyses. The key findings can be summarized as follows:

Micrographs of the intact samples at different magnifications (Figs 6-7) found no individual clay

mineral particles present (with clearly defined boundaries and shapes), although the Old Alluvium is

Figure 9. ESEM micrographs of the DCB-treated M2 sample after the removal of soluble salts. (the particles inside

the ellipse are smectite, while those with an arrow are kaolinite).

known to contains a significant fraction of clay minerals (Table 1). Instead, the clay particles are asso

ciated in a parallel configuration (i.e. face-to-face orientation) within clay pellets of up to 10-20 μm ,

which are covered by a thin coating. These pellets in turn are bridge connected to form relatively

stiff aggregates of 50-100 μm . Moreover, these aggregates are also cemented together to form a

stiff network of the intact soil matrix. However, samples with Fe-oxides removed (Fig. 9) are disin

tegrated and the micromorphology of clay mineral particles is clearly exposed with no coating or bridge

connections. The cementing agents were positively identified as Fe-oxides by SCD, as indicated by the significant

increase in the clay size fraction of a UC sample treated with DCB to remove Fe-oxides (Table 4).

This is further supported by the EDXS analysis of the surfaces of a clay pellet and sand-sized particle

(Fig. 8). Therefore, it can be concluded that Fe-oxides form coatings over clay pellets and form the bridge

connections between pellets and between aggregates. Finally, indirect examination by slaking showed that the intact soil is stable upon immersion in water

and glycerol, indicating that neither the surfaces nor the interlayer spaces of the smectite could be

accessed by water or glycerol molecules to cause swelling (Table 5). However, the DCB treated sample

to remove Fe-oxides does disintegrate and swell in the glycerol solution, indicating that the removal of

Fe-oxides allows the smectite to undergo significant swelling by the expansion of either the double layer

or the interlayer spacing (or both). CEC measurements (Table 5) on samples with and without Fe-oxides

further confirm that the clay particles in the intact soil are coated by Fe-oxides, which significantly

reduce the cation exchange capacity of the clay mineral particles.

4.2.5 A conceptual microstructure model

The application of soil mechanics to practical engineering problems usually requires both conceptual and

mathematical models to describe the behavior of soils (Leroueil & Vaughan 1990). The above summary

has provided a clear picture of the microstructure of the OldAlluvium. Based on its measured mineralogy

and structure, this section introduces a conceptual model of the microstructure in order to understand

the macroscopic engineering properties. It is envisaged that such a conceptual model can form the basis

for building a future constitutive model.

4.2.5.1 A representative structural unit

Figure 10 compares the actual microstructure observed under ESEM with a representative conceptual

microstructural unit, based on a systematic compilation of all the experimental results summarized above.

The conceptual model has the following features associated with the different length scales:

- The particles of the two clay minerals, kaolinite and smectite, are associated in an essentially parallel configuration to form clay pellets of up to 10-20 μm , which are the elementary 'unit' for the intact soil.
- The very fine crystals of Fe-oxides (e.g. the size of goethite is 0.05-0.1 μm , and hematite, 0.02- 0.05 μm) form a coating around the exterior surfaces of the clay pellets.
- A group of clay pellets are further connected together by Fe-oxides to form spherical or semi-spherical, relatively stiff aggregates, 50-100 μm .
- Quartz and feldspar are present as silt and sand size particles, which are also coated and connected to the aggregates by Fe-oxides. These silt and sand particles are distributed in the soil matrix. Since the volume of the clay minerals approaches or exceeds that of quartz and feldspar, and since quartz and feldspar are chemically inert, the overall behavior of the deposit is ultimately dominated by the chemically active clay minerals and Fe-oxides.

- Stiff aggregates, silt and sand particles are again connected together by Fe-oxides to form the stiff intact soil matrix, within which all grains are bound together, and having large inter-connected, inter-aggregate pores (50-80 μm).

4.2.5.2 A micro-mechanical structural model

The presence of smectite indicates that this soil should be susceptible to significant shrinking or swelling

if subjected to drying and wetting. However, the change of the volumetric behavior of smectite in the

intact soil is prevented by the Fe-oxide coating, and significant compression or swelling only occurs after

the soil is loaded beyond the yield stress (σ'_{yp}), which breaks the coating and cementation. The clay pellets

are the elementary structural swelling component for this deposit. That is, any significant macroscopic

swelling should be controlled by the smectite particles within the clay pellets. Similar behavior has been

Figure 10. The resemblance between (a) the microstructure observed under ESEM and (b) a representative

conceptual microstructural unit.

reported on a wide range of naturally occurring soils and weak rocks (Leroueil & Vaughan 1990; 1991),

and it has been attributed to the destructuring caused by compression and swelling and by shearing.

To explicitly take into account such microstructural features, including the surface coating and

particle-level swelling, a conceptual swelling element of microstructure is proposed, which consists

of a precompressed spring and a relatively stiff and brittle shell encapsulating the spring (Fig. 11). The

mechanical spring represents the partially hydrated smectite, while the shell is equivalent to the Fe-oxide

coating. The intact shell carries the initial effective overburden stress (σ'_{v0}) and also prevents pore water

from reaching the smectite particles within the pellets, i.e. the smectite cannot swell by developing a

double layer or through further hydration of interlayer cations. This shell also supports loads that are less

than the yield stress, i.e. when the spring is "inactive".

As the soil is loaded above its in-situ equilibrium condition (i.e. $\sigma'_{vc} > \sigma'_{v0}$) the brittle shell will pro

gressively breakdown enabling the spring to expand as the smectite particles hydrate. During subsequent

unloading, the activated spring can recover the total compression, including the precompression carried

by the shell and the compression induced by the excessive load. After a cycle of loading-unloading, (a) (c) (b) (d) (e)

Figure 11. A micro-mechanical structural model showing: (a) the basic swelling element at the intact state (i.e. under

in situ stress); (b) the element at a compressed state under excessive loading; (c) the element at fully unloaded state

after excessive loading; (d) the mechanical equivalence between the clay pellets and the swelling element; and (e) the

change of volume caused by loading/unloading. L_i , L_c and L_e are the length of the spring at the intact, compressed

and expanded states, while σ'_{v0} and σ'_{vc} are the in situ and applied stresses, respectively. L_e depends on the amount

of coating breakage and resultant expansion of smectite within the clay pellets.

the brittle shell is largely destroyed and the final volume of the basic element can exceed its initial

volume. The swelling behavior of the clay pellet is represented by the proposed conceptual swelling ele

ment. In addition, the aggregates can be represented mechanically by a group of interconnected swelling elements.

It is worth noting the change of the volumetric components of such a micro-mechanical structural

model. The intact soil can be represented by three phases (Fig. 11e): i) volume of solid minerals,

V_s , ii) medium to large inter-aggregate and intra-aggregate voids, V_a , and iii) extremely small voids

within the individual clay pellets, V_p . The voids within the clay pellets include the void between clay

particles/domains and the void within smectite domains, both of which are very small. During initial

compression, the inter/intra-aggregate voids V_a decrease, while those within the pellets (i.e. V_p) remains

largely unchanged. During final unloading after destroying most of the Fe-oxide coating around the

pellets, the smectite now can undergo significant swelling, including particle unbending due to stored

elastic energy, expansion of the electrical double layer on the external surfaces of the smectite, and prob

ably expansion of the interlayer spaces within the smectite. The swelling of the smectite can significantly

increase V_p , which can more than compensate for the decrease in V_a caused by compression.

5 STATE AND INDEX PROPERTIES

In general, residual soils and weathered deposits exhibit index properties which can vary with drying,

remolding, and other physical disintegration processes. An experimental program was conducted to

investigate extensively the variations in the index properties of the Old Alluvium.

5.1 General description of the consistency

During geotechnical site investigation and sampling, the soil's visual appearance, consistency, and plas

ticity were examined by simple identification. The intact samples from both layers have the appearance

of coarse-grained sedimentary soils with varying color. Another striking feature is that the intact sample

blocks are very firm and have the consistency of soft rock, indicating that the soil grains are bonded

together (Figs. 4-5).

There was a significant change in appearance and behavior when a piece of intact soil was pressed

and remolded by hand. The intact soil block was brittle and could be broken down into smaller clumps.

Subsequent pressing and remolding between fingers with increased forces started to disintegrate the

soil clumps, indicating that the soil initially consisted of relatively stiff aggregates. At this stage the

soil acted like a cohesionless material. However, with further pressing and remolding, the soil became

very sticky and plastic, and could even be rolled into small threads, indicating that the relatively stiff

aggregates contained cohesive clays. With increasing stress and continuous remolding, the soil finally

behaved like a plastic clay. The change of the behavior caused by remolding clearly shows that this soil

has a significant sensitivity. It is worth pointing out that the intact soil is only partially saturated. Hence,

changes in consistency are also related to the saturation of the particles.

5.2 Particle size distribution

Particle size analyses were performed on homogenized disturbed samples subjected to increasing degrees

of remolding and different drying conditions (i.e. natural soil without drying, air-drying, and oven

drying). Remolding was achieved by three methods: hand mixing, stirring in a high-speed mechanical

stirrer specified in ASTM D422 (ASTM 1997), and shaking in a mechanical wrist-action shaker. To

vary the remolding energy, the duration of each process was changed.

Figure 12 summarizes the effects of remolding on the particle size fractions of both UC and MZ

specimens of Old Alluvium (data in Table 7). Note that minor hand mixing is reported as zero stirring

time. Clearly, remolding energy has a significant influence on the PSD of the two layers. The clay fraction

increases significantly with stirring time, accompanied by a large reduction of silt fraction, but with little

change of sand fraction. These data indicate that the increase of the clay fraction is caused mainly by

the breakdown of silt-sized aggregates. This is consistent with the microstructure described above (i.e. a

significant portion of the silt-sized aggregates in the natural soil actually comprises bonded clay pellets

containing groups of parallel clay mineral particles).

Figure 13 summarizes the effects of different drying conditions on the particle size fractions with

one-minute stirring (i.e. low remolding energy) of both layers from the data in Table 8. For both layers,

the natural samples have the smallest clay fraction, and air-dried the highest, showing that air-drying

causes more disintegration than oven-drying. Although drying appears to be a simple phenomenon, it

actually involves complex mechanical, physico-chemical, and chemical processes. A detailed discus

sion on the effects of drying can be found in Zhang et al. (2004b). It is also interesting to examine (a) UC layer (b) MZ layer 0 10 20 30 40 50 60 70 0 5 10 15 20 Remoulding - stirring time [min]

F r

a c

t i o

n

[%

] Clay Silt Sand 0 10 20 30 40 50 0 5 10 15 20 Remoulding - stirring time [min] F r a c t i o n [%] Clay Silt Sand

Figure 12. Effects of remolding on the PSD of the natural Old Alluvium.

Table 7. Effects of remolding on the index properties of the Old Alluvium (natural samples only). PSD (%) Atterberg limits (%)

Test No. Remolding energy Clay Silt Sand w L w P I P A c

UC-R1 Only minor hand mixing 40.0 46.2 13.8

UC-R2 ~10 min hand mixing 72.9 40.7 32.2

UC-R3 30 min hand mixing 73.6 37.8 35.8

UC-R4 1 min stirring 44.0 42.6 13.4 (32.2) (0.73)

UC-R5 5 min stirring 53.0 33.5 13.5 82.6 34.8 47.8 0.90

UC-R6 15 min stirring 62.0

UC-R7 20 min stirring 61.5 27.2 11.3 94.3 31.0 63.3 1.03

UC-R8 15 hour shaking 56.5 29.4 14.1

MZ-R1 Only minor hand mixing 8.5 45.5 46

MZ-R2 ~10 min hand mixing 44.7 22.5 22.2

MZ-R3 30 min hand mixing 44.9 21.4 23.5

30

40

50

60

70

80 0 10 20 30 40 50 60 70 80 90 100 Liquid limit, w L [%]

P l

a s

t i c

i t y

i n

d e

x ,

I P

[%

] UC MZ U-Line A-Line 0 10 20 30 40 50 60 70 80 0 10 20 30
 40 50 60 70 80 90 100 Liquid limit, w L [%] P l a s t i c
 i t y i n d e x , I P [%] U-Line A-Line UC MZ (b)(a)
 Natural Air drying Oven drying

Figure 14. Effects of (a) remolding and (b) on the Atterberg limits of the natural Old Alluvium.

5.3 Atterberg limits

Figure 14a shows the effects of remolding energy (arrows indicate increasing remolding energy) on the

Atterberg limits in a Casagrande plasticity chart (data from Table 7). Both the liquid limit (w L) and

plasticity index (I P) increase with remolding energy, while the plastic limit (w P) decreases. Furthermore,

the data for both UC and MZ layers move away from the A-line, showing that remolding increases the

toughness (i.e. the ratio of plasticity index to liquid limit) of the soil. The higher plasticity of the UC

layer relative to the MZ reflects its higher content of clay minerals (54% versus 38%; Table 1).

Table 8 and Fig. 14b show the Atterberg limits measured on the samples subjected to different drying

conditions. Compared to the natural soil, drying causes an increase in I P and a decrease in w P for both

layers, while w L either decreases slightly (UC) or increases, especially after air-drying (MZ). For both

layers, air-drying produced the highest I P , but for different reasons (lowest w P versus highest w L). In any

case, the overall effects of drying are modest compared to those of remolding due to the relatively low

(but nevertheless standard) remolding energy.

5.4 Activity

It is of interest to examine the effects of remolding energy and drying on the activity (A c) of the Old

Alluvium. All tests shown in Table 7 were performed on natural samples. Tests UC-R2 and MZ-R2 used

the hand mixing energy specified in the ASTM standard method for measuring Atterberg limits, while

UC-R4 and MZ-R4 used the one-minute stirring energy specified in the standard method for particle

size analysis. The corresponding values of A c are shown with brackets in Table 7. As expected, greater

remolding significantly increases the activity of both layers. Although UC has a higher clay mineral

fraction than MZ, the activity of the UC material is only half that of the MZ. This is attributed to the

higher amount of Fe-oxides in the UC layer. The Fe-oxides have positive surface charges in an acidic

environment (i.e. pH < 7, as shown in Table 2) and hence,

are attracted to the negatively charged clay

particles (Schwertmann & Taylor 1989). This causes a reduction of the surface reactivity of the smectite

and hence a lower activity.

Table 9. Effect of sample preparation methods on the PSD of the Old Alluvium. Clay Silt Sand

Sample Initial sample condition Remolding energy (%) (%) (%)

UC Natural 15-20 min stirring 62.0 27.2 11.3 Air-dried 1 min stirring 62.5 24.0 13.0 Oven-dried 20 min stirring 60.0 26.9 13.1 Mineralogical analysis 63

M2 Natural 20 min stirring 37.0 24.2 38.8 Air-dried 1 min stirring 29.5 28.1 42.4 Mineralogical analysis 41

Table 10. Summary of the initial conditions of intact block samples. Sample location Depth Density Water Content No.

Soil layer (Fig. 2) (m) (g/cm³) (%) measurements

UC (a) B3 1.2 1.81 ± 0.09 27.03 ± 0.15 3 (b) B7 6.0 1.73 ± 0.04 39.03 ± 2.50 15

M2 (c) B1 10.7 2.08 15.60 1 (d) B2 12.0 1.98 ± 0.03 13.65 ± 0.80 7 Table 9 shows that drying causes a reduction in A c , with the effect being more pronounced for air

drying than oven-drying. The relatively low remolding energy (one-minute stirring) used for these tests

obviously was more effective in increasing the clay size fraction than increasing I P . Apparently the

dispersion of previously aggregated clay pellets of 10-20 µm did not cause a corresponding dispersion

of the clay particles (<2 µm) contained within these pellets. Hence, the full plasticity of the smectite

particles is still largely masked by the Fe-oxide coating on the pellets.

5.5 Intrinsic particle size distribution

As the natural Old Alluvium consists of silt-sized aggregates comprising clay pellets cemented together

by Fe-oxides, it is of interest to know its intrinsic particle size distribution and the maximum clay fraction

obtained by various sample preparation methods. This requires all aggregates to be dispersed to individual

particles (or at least to diameters of less than 2 μm). Table 9 summarizes the particle size fractions

observed by different sample preparation methods. The maximum clay fractions are approximately

62% and 37% for the UC and MZ, respectively (obtained by stirring natural samples for 20 minutes).

However, for the UC, the clay fraction after air-drying is nearly equal to that by stirring the natural and

oven-dried samples (Table 4), indicating that it is possible to achieve the same degree of dispersion (or

at least the same clay fraction) with different sample preparation methods. Table 9 also includes the

clay size fractions obtained by quantitative mineralogical analyses, which are surprisingly consistent

with the results of particle size analyses. Therefore, it can be concluded that the intrinsic particle size

distribution of this Old Alluvium can be obtained by remolding intensively the natural samples or using

less remolding with the air-dried samples.

6 ENGINEERING PROPERTIES

The engineering properties of the Old Alluvium were mainly obtained by laboratory oedometer and

triaxial tests. As stated previously, high-quality block samples were obtained from the UC and MZ by

taking the advantage of open excavations during tunnel construction (Fig. 2). All block samples were

assessed by transmission X-ray radiography and only those blocks showing no cracks or detectable

disturbance were chosen for laboratory testing. In addition, special attention was paid to selecting the

more homogeneous parts from the large block to measure the strength and deformation properties,

since the material contains irregularly distributed white veins (especially for the UC layer). Table 10

summarizes the initial conditions of the intact block samples.

Table 11. Initial specimen conditions of the four oedometer tests.

Test ID	Sample	e_0	S_0 (%)	σ'_v (kPa)
---------	--------	-------	-----------	-------------------

Oed07	UC, B3	0.86	83	40
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Oed11	UC, B3	0.80	91	40
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Oed13	UC, B3	0.90	80	40
-------	--------	------	----	----

Oed08	MZ, B1	0.48	86	160-180
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Table 12. Summary of control parameters for all triaxial tests. Back Pre-shear Post-shear pressure confining pressure water content

Test ID	Sample	Stress path *	σ'_v (kPa)	B value	(%)
---------	--------	---------------	-------------------	---------	-----

TX511	UC, B7	C(U)	280	0.87	200 44.0
-------	--------	------	-----	------	----------

TX512	E(U)	280	1.02	200	40.2
-------	------	-----	------	-----	------

TX514	E(L)	296	0.87	184	39.6
-------	------	-----	------	-----	------

TX518	C(U)	276	0.84	204	44.3
-------	------	-----	------	-----	------

TX522	E(U)	270	0.84	180	45.3
-------	------	-----	------	-----	------

TX524	C(L)	244	0.97	196	34.6
-------	------	-----	------	-----	------

TX525	C(L)	253	1.01	187	36.3
-------	------	-----	------	-----	------

TX507	MZ, B2	C(L)	278	0.51	168 17.9
-------	--------	------	-----	------	----------

TX508	E(U)	277	0.63	173	17.7
-------	------	-----	------	-----	------

TX509	E(L)	250	0.62	190	17.9
-------	------	-----	------	-----	------

* C = compression; E = extension; L = loading; U = unloading.

The consolidation behavior was measured through one-dimensional incremental loading tests by

combining a conventional oedometer with a universal loading frame featuring an advanced feedback

control system. The loading frame can automatically apply and accurately control loads ranging from

0.5 kg to 30,000 kg, and can hold a constant stress. The test specimens are 6.35 cm in diameter and

2.0 cm thick. The highest stress applied to the specimen was nearly 45 MPa. To minimize the effect

of the side friction on the specimen during both compression and swelling, a stainless steel ring with

highly polished inner surface coated with vacuum grease was used to contain the specimen. The effects

of destructuring caused by pre-stressing have been examined by performing unloading-reloading cycles

at various stress levels. Table 11 shows the initial specimen conditions of four consolidation tests.

Triaxial shear tests were performed using the MIT automated triaxial system featuring an on-specimen

small strain measurement device (Sheahan et al. 1990, Da Re et al. 1999). All tests used the standard

specimens that are 8 cm high and 3.5 cm in diameter and were trimmed from intact block samples. The

main objective of these tests was to measure the strength and deformation characteristics of the intact soil

(i.e. preserving the initial structure as much as possible before actual shearing) in a saturated condition.

This was achieved by applying a small hydrostatic confining pressure which was sufficient to prevent

the specimen from swelling during back-pressure saturation

(balancing the swelling pressures observed

during inundation of oedometer test specimens) while causing minimal compression of the specimens

(in their natural partially saturated condition). After a few trials, the pre-shear confining pressure was

optimized as ~200 kPa and ~180 kPa for the UC and MZ, respectively. These are very low stress levels

compared to the yield stresses observed in the oedometer tests (cf. Fig. 15) and was applied in increments

of ~40 kPa before the saturation process was started. The saturation process was carefully monitored for

each test by observing the changes of the axial strain once saturation was started. If swelling occurred,

the saturation process was immediately stopped and the confining pressure was increased to prevent

further swelling strains. Table 12 lists the control parameters used in the triaxial test program.

In summary, all triaxial tests were performed by following the procedures listed below:

- Increase without drainage the confining pressure by steps to the desired pre-shear effective target

stress described above;

- Saturate without volume change the specimen;
- Perform drained shear at a constant strain rate of 0.15%/hour along a variety of prescribed effective

stress paths. -5 0 5

10

15

20

25

30

```

35 1 10 100 1000 10000 100000 Consolidation stress,  $\sigma'$  vc
[kPa]

V e

r t i

c a

l s

t r a

i n

,

e

[ %

] 1st loading 1st unloading 2nd loading 2nd unloading -4 -2
0 2 4 6 8

10

12 10 100 1000 10000 Consolidation stress,  $\sigma'$  vc [kPa]

V e

r t i

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l s

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] 1st loading 1st unloading 2nd loading 2nd unloading 0 5
10 15 20 25 30 35 10 100 1000 10000 100000 Consolidation
stress,  $\sigma'$  vc [kPa] V e r t i c a l s t r a i n , e [ % ]
1st loading 1st unloading -10 -5 0 5 10 15 20 25 10 100
1000 10000 100000 Consolidation stress,  $\sigma'$  vc [kPa] V e r

```

vertical strain, ϵ [%] 1st loading 1st unloading
 2nd loading 2nd unloading 3rd loading 3rd unloading (a)
 Oed07, UC; $\sigma'_{v0} = 40$ kPa, $\sigma'_p = 800$ kPa. (b) Oed13,
 UC; $\sigma'_{v0} = 40$ kPa, $\sigma'_p = 800$ kPa. (c) Oed11, UC; σ'_{v0}
 $= 40$ kPa, $\sigma'_p = 800$ kPa (d) Oed08, M2; $\sigma'_{v0} = 160$ - 180
 kPa, $\sigma'_p = 6$ MPa

Figure 15. Consolidation curves for the four oedometer tests.

6.1 Compressibility

Figures 15a-d illustrate the compressibility of the Old Alluvium through plots of vertical strain ϵ at the

end of primary consolidation against vertical consolidation stress σ'_{vc} in four oedometer tests. These

results show a number of important features of behavior. Although the specimens exhibit very stiff behavior after inundation at their characteristic in situ stress

levels, there is a well defined yield stress producing a high apparent overconsolidation. The three UC

tests (Figs. 15a-c) all exhibit $\sigma'_p \approx 800$ kPa, implying an apparent OCR ≈ 20 , while $\sigma'_p \approx 6$ MPa for

the M2 specimen (OCR ≈ 35 , Fig. 15d). The Fe-oxide cementation is undoubtedly responsible for the

high stiffness of the intact specimens. The yield stress is most probably related to the crushing of the

aggregates and silt-sand particles (i.e. quartz and orthoclase) but may also be influenced by other well

known overconsolidation mechanisms associated with the geological history (such as erosion, Velde

1995). The compression behavior can be viewed through the framework of compressibility for granular

materials proposed by Pestana and Whittle (1995). The M2 soil exhibits a much higher yield stress due

to the increased fraction of sand particles (49% vs. 31% for UC of quartz and orthoclase) which have a

higher crushing strength than the aggregates (made up of

clay pellets). The inter-aggregate cementation produces only a modest cohesive shear strength and is almost certainly broken down at stress levels below the observed yield stress. For specimens loaded above the initial yield stress, subsequent unloading produces large swelling of the Old Alluvium. In some cases (e.g. Figs. 15a, d) there is even a net increase in the volume compared to the original intact state. The magnitude of the swelling strain (or swelling ratio, SR) increases with the imposed level of preconsolidation pressure. This type of behavior reflects the progressive damage associated with the breakdown of the aggregates and can be explained from the conceptual microstructural model presented above. As the intact soil is subjected to stresses beyond yielding, however, Fe-oxide aggregate coatings are broken down, resulting in the exposure of smectite to inter-aggregate pore water. Subsequent unloading allows water to enter the interlayer spaces producing crystalline swelling of the smectite (Likos & Lu 2006). This postulated mechanism has been partially validated using conventional SEM micrographs on (air-dried) specimens that have been precompressed and then allowed to swell freely.

Figure 16. SEM micrographs of compressed samples: (a) a horizontal plane taken from an oedometer specimen of the UC; (b) a horizontal plane taken from an oedometer specimen of the MZ.

(Fig. 16). The SEM results are affected by both drying and compression (and their effects cannot be separated). However, it is clear that samples which have been compressed possess a relatively dispersed structure with parallel orientation of particles. When compared with the intact structure obtained by

ESEM (Figs. 6-7), there are no large inter-aggregates pores in the compressed samples. Furthermore,

the boundaries between platy clay particles become more visible (similar to effects of SCD treatment

in Fig. 9), indicating that Fe-oxides cementation has been broken by mechanical compression and

subsequent swelling. Again, similar behavior was described by Leroueil & Vaughan (1990).

Reloading of the oedometer test specimens (Fig. 15) produces a large hysteresis in compression

response, with almost perfect recovery of the pre-consolidation state (stress and void ratio). This can be

seen most clearly by comparing the 1st unload and 2nd loading segments in Figure 15a, c. This behavior

is consistent with theoretical modeling of crystalline swelling and compression of smectite (Boek et al.

1995).

The compression behavior of the OldAlluvium in the 'normally consolidated' regime can be described

by the Limiting Compression Curve (LCC) proposed by Pestana and Whittle (1995). The compressibility

is described by a parameter p_c (linearization in $\log e - \log \sigma'_{vc}$ space). Zhang et al. (2003) have reported

$p_c = 0.26-0.27$ for the UC, which is similar to other high plasticity clay, while $p_c = 0.47$ for the M2 mate

rial (Fig. 15d) is comparable to the crushing response of ground quartz. More recent tests by Nikolinakou

(2006) have shown that the compressibility parameter, p_c , increases with stress level suggesting that

progressive breakdown of the aggregates continues even at very high pressures (exceeding 50 MPa).

6.2 Rate of consolidation

Figure 17a shows the vertical coefficient of consolidation, c_v , from one of the incremental oedometer

tests (oed07, UC, Table 11). Here c_v was determined based on each incremental loading curves using

the Casagrande (or Log Time) Method. The results show a remarkable decrease in the consolidation

coefficient with confining pressure, from $c_v \approx 0.1 \text{ cm}^2/\text{s}$ at 80 kPa to 3×10^{-5} at 20 MPa (i.e. a change

of almost four orders of magnitude).

This behavior is further illustrated through comparisons of the displacement-time curves observed

during application of a single load increment (400-800 kPa) in first and second loading stages (oed11,

UC, Table 11) (Fig. 18a). The time for end-of-primary consolidation t_P increases by two orders of

magnitude for the loading stages. Figure 18b shows that the swelling strain for the second unloading is

much greater than that of the first unloading, over the same increment of stress (50 to 12.5 kPa).

The observed changes of c_v are very different from the stereotypical behavior of sedimentary clays

which generally show $c_v \approx \text{constant}$ in the normally consolidated regime. A second remarkable feature

of the data in Figure 17 is the continued decrease in c_v during unloading/swelling stages of the test. The

reversal of loading direction is associated with a step increase in stiffness of the soil skeleton and hence,

net reductions in c_v can only be explained by continued decreases in the hydraulic conductivity of the

specimen during swelling.

In order to explain these unusual phenomena, it is again helpful to use the conceptual soil microstruc

tural model. The intact material possesses a dual porosity with large inter-aggregate pores and a cemented (a) Consolidation coefficient for UC specimen Oed07. (b) Comparison of consolidation coefficient for UC and MZ specimens. 0.000001 0.00001 0.0001 0.001 0.01 0.1 1 10 100 1000 10000 100000 Consolidation stress, σ'_{vc} [kPa] C o e f f i c i e n t o f c o n s o l i d a t i o n , c_v [c m ² / s] Oed07 Oed08 Oed11 Oed13 0.00001 0.0001 0.001 0.01 0.1 1 10 100 1000 10000 100000 Consolidation stress, σ'_{vc} [kPa] C o e f f i c i e n t o f c o n s o l i d a t i o n , c_v [c m ² / s] 1st loading 1st unloading 2nd loading 2nd unloading

Figure 17. Consolidation properties of Old Alluvium. (a) (b) 0 1 2 3 4 1 10 100 1000 10000 100000 Time [sec]

D

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t [0

.

0 1

c m

] 1st loading from 400 to 800 kPa 2nd loading from 400 to 800 kPa -7 -6 -5 -4 -3 -2 -1 0 1 10 100 1000 10000 100000 1000000 Time [sec] D i s p l a c e m e n t [0 . 0 1 c m] 1st unloading from 50 to 12.5 kPa 2nd unloading from 50 to 12.5 kPa

Figure 18. Examples of incremental compression and swelling curves from Oed11: (a) Loading from 400 to 800 kPa;

(b) Unloading from 50 to 12.5 kPa.

skeleton (accounting for the high initial coefficient of

consolidation). During loading, the compressive stress destroys the cementation bonds between aggregates and breaks the coatings over clay pellets, allowing access to water and interlayer-swelling of smectite. As the clay pellets swell, they tend to infill the macro-pores producing a large decrease in hydraulic conductivity. Upon unloading, the consolidation stress decreases and hence smectite particles are allowed to swell further and to move and re-orient more freely, resulting in further reduction in macro-pores and hydraulic conductivity. After unloading, the soil possesses a different microstructure with a great reduction of void space and in apparent average grain size (Fig. 16). Subsequent reloading to a higher stress can breakdown more cementation bonds and cause the same effect as the previous loading. Consequently, c_v is dependent upon the number of unloading cycles.

Figure 17b shows that the UC and MZ specimens undergo similar changes in consolidation properties.

However, it is interesting to note that test oed13 (UC, Table 1) has much higher values of c_v (at the same confining pressure) than the other three oedometer tests. This result occurs because it is the only one of the four oedometer tests that did not include a sequence of unloading and reloading stages (Fig. 15).

Although the compressibility of oed13 is similar to the other tests on UC specimens at high pressures (LCC regime), there is less reduction in the hydraulic conductivity as much less particle re-orientation (and pore infilling) have occurred. Hence, the results in Figure 17b confirm the importance of the stress

history on both the consolidation and flow properties of the Old Alluvium.

6.3 Intact shear strength

Figures 19a, b shows the stress paths (Note: $p' = (\sigma'_{v} + \sigma'_{h})/2$ and $q = (\sigma'_{v} - \sigma'_{h})/2$) for the suite of

drained shear tests performed on the intact UC and MZ samples, respectively, while Figure 20 show the

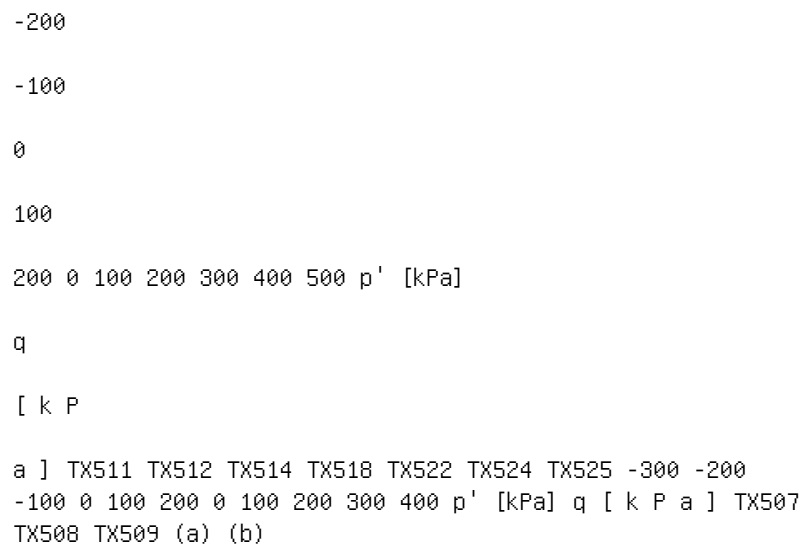
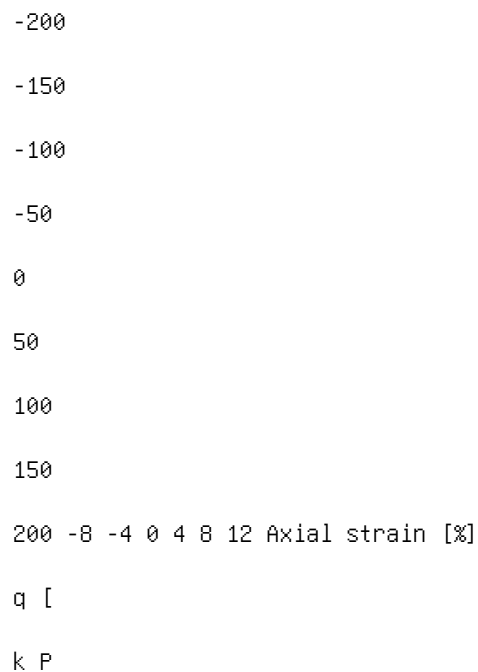


Figure 19. Stress paths for the triaxial drained shear stress (a) The UC layer; (b) The MZ layer.



c] TX511 TX512 TX514 TX518 TX522 TX524 TX525 -300 -250
 -200 -150 -100 -50 0 50 100 150 200 -4 -2 0 2 4 Axial
 strain, ϵ [%] q [k P a] TX507 TX508 TX509 (a) (b)

Figure 20. Stress-strain curves for the triaxial drained shear tests: (a) The UC layer; (b) The MZ layer.

Table 13. Strength parameters measured in drained triaxial shear tests on intact specimens. p' f q f a' α' c' ϕ'

Sample	Test ID	(kPa)	(kPa)	(kPa)	($^\circ$)	(kPa)	($^\circ$)	r^2
UC	TX518	106	67	22.4	23.3	25	25.5	0.9888
	TX511	146	83	TX525	234	118	TX524	397
	TX512	128	-75	TX522	151	-97	TX514	363
								-174
MZ	TX507	189	138	17.5	32.5	23	39.5	0.9999
	TX508	111	-88	TX509	378	-258		

Notes: (1) Failure envelope is defined as $q = a' + p' \tan \alpha'$ in MIT p - q space.

(2) $\tan \alpha' = \sin \phi'$, $c' = a' / \cos \phi'$.

stress-strain curves from these tests. Key parameters are summarized in Table 13. The shear strength

of the intact Old Alluvium can be well described by the conventional Mohr-Coulomb envelope. The

cohesive and frictional components of strength have been obtained by linear regression and are practi

cally identical in the compression and extension modes of shearing. This apparent strength isotropy may

be attributed to the Fe-oxide cementation which exhibits no preferred directions and forms a randomly

oriented stiff cemented-aggregate network. In addition, tropical weathering tends to remove the original

fabric (inherent anisotropy) that existed due to the original alluvial sedimentation. Similar behavior was

discussed by Leroueil & Vaughan (1990). The intact UC and MZ have approximately the same cohesion $c' = 23 - 25$ kPa, while the intact MZ

has a much higher friction angle ($\phi' = 39.5^\circ$) than the UC layer ($\phi' = 24.2^\circ$). This is due to the fact that

MZ is much denser than UC (Table 10) and contains a greater percentage of coarse-grained particles

(quartz and orthoclase). Although differences in the frictional component of strength are expected, it is

perhaps surprising that the cohesion is so similar for UC and MZ specimens given the differences in

constitution and fractions of Fe-oxides. It is worthwhile pointing out that strength variations do exist in the UC due to the complex patterns

of white veins which were introduced to the deposit through leaching of Fe-oxides along old cracks

and root paths. Details on such macro-structural heterogeneity can be found in the previous section on

macrofabric. A plane containing a greater percentage of white veins usually has a weaker strength. If it

happens to coincide with the failure surface, then the measured shear strength may be lower than that

of the intact soil matrix (Figs. 19-20). Since the white veins are not so well developed in the MZ, the

in situ strength variation in the MZ is expected to be much smaller. One more particular feature is that

the intact soil, due to the Fe-oxide cementation, shows a much brittle behaviour, as indicated by the

relatively small failure strain.

7 ENGINEERING PROBLEMS

The following paragraphs outline some of the known and potential engineering problems associated with

the Old Alluvium.

7.1 Identification and classification

Iron oxides cause cementation and aggregation of clay particles in the Old Alluvium. This causes dif

difficulty in field identification and classification of the soil, since the aggregated clay particles look like

sand or silt grains and are quite stiff and stable (referred to as a 'pseudo-silt' by soil scientists). Simple

visual/manual inspection of the intact Old Alluvium indicates that it is composed of silt and sand-sized

grains with little or no plasticity. This initial assessment can be very misleading since progressive remold

ing causes large increases in both the clay size fraction and plasticity of the soil (e.g. Table 7, Figs. 12,

14). Hence the results of routine classification tests on similar residual soils can vary significantly with

the precise techniques used for sample preparation. The degree of structure possessed by intact soil can

(and should) be assessed, at least roughly, by varying the preparation techniques for classification testing.

Particle size distribution (PSD) and Atterberg limit tests form the basis for the Unified Soil Classifica

tion System (USCS) and are widely used in empirical correlations for estimating engineering properties.

The standard sample preparation techniques that have been established for these tests provide consistent

and generally reliable results for typical cohesive sedimentary soils if the standard remolding energy

produces a homogeneous fabric primarily composed of individual soil particles (i.e. a highly dispersed

structure with minimal particle aggregation). Note, however, that Atterberg limit tests for some soils

(e.g. those containing organic matter, smectite, hydrated halloysite, or a high concentration of soluble

salts in pore fluid) should start from the natural water content since pre-drying can drastically reduce

their plasticity. In contrast, the interpretation and use

of index properties for highly structured soils,
especially those with significant cementation becomes
problematic since PSD and Atterberg limits are
strongly affected by the precise sample preparation
technique. For the cemented Old Alluvium, the clay
size fraction and plasticity increase significantly with
increasing remolding energy beyond the ASTM
specified standard. These variations are sufficient to
alter the basis USCS classification of the UC layer
from MH to CH and the MZ layer from CL to CH; the activity
also increased. For the standard energy,
pre-drying caused a relatively modest increase in I_P ,
compared to the clay size fraction and hence, a
decrease in activity. Moreover, the intact soil (i.e. no
remolding, drying or loading beyond the yield
stress) behaved as a stiff, brittle, essentially
non-plastic material with a very high hydraulic
conductivity.
Hence, the appropriate sample preparation technique for
index testing and proper interpretation of the
results for this type of residual soil and similar deposits
depends on how the soil will be used in practice.
At one extreme, if old Alluvium is used as structural fill
compacted wet of optimum water content
(to minimize volume change due to long-term wetting), then
index tests on highly remolded samples
are appropriate since seasonal cycles of wetting and drying
can cause significant damage to structures
supported on high plasticity fills. At the other extreme,
the results from index tests on natural soil with
standard remolding will give a misleading indication of the
in situ behavior of lightly loaded, undisturbed
intact soil. For practical purposes, index tests should be
performed at varying remolding energies (with

and without pre-drying) in order to establish the stability of the microstructure and potentially adverse

changes in engineering properties due to disturbance (damage caused by drying, shearing, etc.) during

construction.

7.2 Undisturbed sampling

Undisturbed sampling of the Old Alluvium is very difficult and thus best practice should obtain large

block samples rather than tube samples. Standard sampling practice in Puerto Rico at the start of the

current research used driven thick-walled samplers with a plastic internal sleeve. Transmission X-ray

radiography showed that these 10 cm diameter tube samples were too poor to be used for meaningful

strength and consolidation property measurements. Sample disturbance appeared to break the inter

aggregate cementation, causing the material to flow and hence, low recovery rates were observed.

7.3 Laboratory testing and soil properties

The handling and preparation of test specimens for laboratory testing of the Old Alluvium also requires

considerable attention to detail (e.g. controlled humidity to minimize air-drying, use of carbide tipped

trimming blades to handle abrasive quartz particles).

Progressive disturbance and remolding of intact soils similar to the Old Alluvium can be expected to

produce significant changes in engineering properties, including: 1) a decrease in shear strength; 2) an

increase in compressibility during in 1-D compression testing; 3) a large decrease in hydraulic conduc

tivity; and 4) an increase in shrinkage and swelling during cycles of drying and wetting. These changes

indicate that disturbance of the intact soil may lead to unexpected problems during both underground

(e.g. excavations and tunnels) and above ground (e.g. shallow foundations) construction. Also, residual

soils such as the Old Alluvium should only be used with great caution for compacted structural fills.

7.4 Design and construction

The spatial variability of the Old Alluvium (due to its original alluvial deposition, weathering, Fe-oxides

cementation and gleying) presents a major challenge in estimating mass parameters for design. The

macro-fabric includes potential planes of weakness (i.e. white veins), while the vertical profile (Fig. 2)

is characterized by a transition from highly weathered, moderately cemented surficial units (UC) to

unweathered cohesionless alluvial soils (LS). Underground construction of the Tren Urbano in Río Piedras involved a variety of construction

methods including cross-lot braced excavations, EPB bored tunnels, NATM tunneling (supported by

shotcrete and lattice girder) and open face excavation of a series of stacked drifts forming the structural

support for the station cavern (Whittle & Bernal 2003). In order to control the effects of tunnel-induced

ground movements on the overlying buildings, the projects also made extensive use of a compensation

grouting system. The project encountered major difficulties in controlling the ground movements most notably for the

construction of the stacked drifts and EPB bored tunnel sections where maximum surficial settlements

exceeded 150 mm and 120 mm, respectively. In general the performance of the NATM and braced

excavations were more satisfactory. It is difficult to

assess the underlying causes for these unexpectedly

large ground movements. However, it is clear that the use of compensation grouting schemes (in the

UC and MZ zones) and control of face pressures in the EPB machine (including conditioning of the

excavated material through the screw conveyor) did not take into account the potential deleterious effects

of remolding the Old Alluvium. Problems of the stacked drift construction were most pronounced for the

deep drifts located at the interface between the LS and MZ zones, where face stability was marginal. The

cutting tools for the tunnel boring machine also experienced high rates of erosion for tunneling through

the highly abrasive MZ materials. Working conditions inside the excavations deteriorated markedly

in wet conditions when (the air dried, and remolded) Old Alluvium became very sticky (reducing the

efficiency of excavation and transport operations).

8 CONCLUSIONS

This paper has synthesized the results of an extensive study to understand the composition, microstructure

and engineering properties of the Old Alluvium from a tunneling project in San Juan, Puerto Rico.

At this site, the vertical profile includes more than 25 m of Pleistocene-age Old Alluvium, ranging

from an intensively weathered Upper Clay (UC) unit, through a less weathered, Middle Zone (MZ)

to unweathered Lower Sand (LS) layers. The weathering process has produced a combination of stable

primary minerals (quartz and orthoclase, which constitute 31% and 49% of the UC and MZ, respectively),

clay minerals (kaolinite and two species of smectite) and Fe-oxides (hematite and goethite) forming a

complex microstructure. A systematic synthesis of the experimental results (for UC and MZ materials)

has found that the intact soil consists of a stiff network of interconnected aggregates (50-100 μm)

with large inter-aggregate pores. The aggregates comprise clusters of clay pellets (10-20 μm), which

in turn are made up of clay particles that are arranged in a parallel (face-to-face) configuration. The

very fine Fe-oxides, which constitute less than 5% to 10% of the solid fraction, play two critical roles

in controlling the structure of the intact soil: firstly, they form an impermeable coating around the

clay pellets, which negates the inherent activity of the clay mineral particles. Secondly, they provide

cementation bonds that hold the pellets together to form stiff spherical aggregates and also interconnect

the aggregates and other silt and sand-sized particles to form a very stable and relatively stiff intact

soil matrix. The intact Old Alluvium is very stable (and does not slake), has a high bulk hydraulic

conductivity, and exhibits an effective cohesion $c' \approx 25$ kPa (for both UC and MZ), while the frictional

component of shear strength is related to the fraction of primary granular minerals ($\phi' = 26^\circ$ in UC vs.

40° for MZ). The Authors have proposed a conceptual microstructural model which provides a qualitative frame

work for explaining the relationship between macroscopic engineering properties and degradation of this

microstructure. Mechanical (or chemical) breakdown of the Fe-oxide coatings and cementation bonds

enables the (partially dehydrated) clay minerals to imbibe water, greatly increasing the plasticity and

clay-sized fraction. These processes produce large changes

in the 1-D consolidation behavior of the Old

Alluvium. As the preconsolidation stress increases there is a dramatic increase in the observed swelling

strains upon unloading (attributed principally to crystalline swelling of the smectite). The consolidation

coefficient reduces during first loading and continues to decrease during unloading. This behavior can

be linked to the closure of macro-pores and a massive reduction in the bulk hydraulic conductivity. Underground construction of the Tren Urbano through the Old Alluvium in Río Piedras did encounter

considerable difficulties in controlling ground movements. Some of these problems have been directly

attributed to mis-classification and interpretation of the soil profile. However, other processes, such

as conditioning of the soil excavated by the EPB tunnel boring machine and compensation grouting,

would have caused substantial destructuring of the Old Alluvium and may also be important factors that

contributed to the unexpectedly large ground movements having occurred during tunneling.

ACKNOWLEDGMENTS

This study was supported by Tren Urbano in conjunction with the underground construction of Section 7

of the Tren Urbano System in San Juan, Puerto Rico. Partial funding was also provided by NSF through

Grant No. CMS-0085397. The Authors are also grateful to Prof. C. C. Ladd and Dr. R. T. Martin for

their technical help with details of soil behavior and clay mineralogy.

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Zhang, G., Germaine, J.T., Whittle, A. J. & Ladd, C. C. 2004b. Index properties of a highly weathered Old Alluvium. Géotechnique 54(7): 441-451. Methane hydrate soils Characterisation and Engineering Properties of Natural Soils - Tan, Phoon, Hight & Leroueil (eds) © 2007 Taylor & Francis Group, London, ISBN 978-0-415-42691-6

Characterisation and engineering properties of methane hydrate soils

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ABSTRACT: Methane hydrate soil is a natural soil deposit that contains methane hydrate in its pores.

Methane hydrate is a metastable solid material and it bonds the soil particles together. Methane hydrate

soil can only develop and exist under a condition of high pressure and low temperature. Hence, natural

methane hydrate soils are usually found under deep seabed or permafrost regions. This paper synthe

sises the available data of engineering properties of natural methane hydrate soil samples retrieved at

four different sites; Nankai Trough, Mallik-Mackenzie Delta, Blake Ridge and Hydrate Ridge. The

geotechnical data are obtained from index tests, oedometer tests, triaxial compression tests and wave

velocity measurements. The effects of hydrate growth pattern and hydrate saturation on their mechanical

properties are discussed.

1 INTRODUCTION

Methane hydrate soil is a natural soil deposit that contains methane hydrate in its pores as shown

schematically in Figure 1. Methane hydrate, which is one of the forms of gas hydrate, is a metastable

solid material that consists of methane (CH_4) and water molecules. As shown in Figure 2, the water

molecules form a structure, in which a floating molecule of methane gas is caged. For the structure

shown, 1 m³ of methane hydrate can release 164 m³ of

methane gas and 0.87 m³ of water when the

hydrate dissociates (i.e. becomes unstable) under standard condition.

Figure 1. Distribution of methane hydrate in

methane hydrate soil. Figure 2. Example of molecular structure of methane hydrate, in which methane gas molecule is surrounded by water molecules. P r e s s u r e (a t m)
Temperature (°C) 100 1000 10 -10 -5 0 5 10 15 20 25

Figure 3. Methane hydrate phase equilibrium relation (after Kamath, 1984).

Figure 4. Variation of temperature (T) and pressure (P) in permafrost and oceanic regions. The increasing temperature

with depth in soil is termed as geothermal gradient which is commonly 2-6 °C/100 m and the lower limit of methane

hydrate zone is marked by Bottom Simulating Reflector (BSR). The gray line is the phase equilibrium of gas hydrate. The stability of methane hydrate depends on pressure and temperature as shown by the phase equi

librium relationship in Figure 3. Methane hydrate soil can only develop and exist under a condition of

high pressure and low temperature (for example, 100 kPa at -80 °C and 2.5 MPa at 0 °C. Hence, natural

methane hydrate soils are usually found under deep seabed (i.e. 1 to 2 km deep from the sea level and few

hundreds of metres below deep sea floor) or permafrost regions as shown in Figure 4. At such locations,

the existence of high pressure and low temperature condition allows formation of methane hydrate in

soil pores. Figure 5 shows worldwide locations of known and inferred methane hydrate soils in oceanic

and permafrost regions. Methane hydrate soil is highly compacted (i.e. stable) under deposited conditions and is likely to

behave as a bonded sedimentary soil. However, once the hydrate dissociates (i.e. becomes unstable) due

to decrease in pressure and/or increase in temperature, a large volume of methane gas is released causing

significant changes in pore pressures. Of particular scientific and engineering interests that have led studies of methane hydrate soils are;

(a) submarine geohazard, (b) future potential energy resource and (c) global climate issues. Decrease in

pressure and increase in temperature due to sea level changes lead to dissociation of methane hydrate,

initiating submarine landslides (Bugge et al., 1987; Kayen & Lee, 1991; Dillon et al., 1998; Haq, 1998).

Decrease in pressure also occurs during drilling of an oil/gas well. If the well is drilled into a methane

Figure 5. Worldwide distribution of confirmed (open circle) and inferred (solid circle) gas hydrate-bearing sediments

(after Kvenvolden & Lorenson, 2000).

hydrate soil layer, the sudden increase in volume by gas generation can potentially damage the well

(Yakushev & Collett, 1992; Collett & Dallimore, 2002).

By dissociating the methane hydrate in controlled manner, methane gas can be extracted. It is estimated

that the methane hydrate accumulated in the subsurface holds more than 10 trillion tons of carbon, which

is twice as much as the current world's coal, oil and conventional gas reserves combined (Kvenvolden,

1998). Consequently, methane hydrate is considered as a potential fuel of the third millennium as it

offers a substantial source for the ever-increasing demand for energy.

Natural dissociation of methane hydrate leads to release of methane into the atmosphere. Methane

is a greenhouse gas and its global warming potential is 20 to 40 times larger than an equivalent weight

of carbon dioxide (i.e. Kvenvolden, 1998). However, its exact impact on the greenhouse effect is still

unclear, but it is a function of the rate of methane gas released. A slow release of methane gas over

geological time will slow the oxidation to carbon dioxide, preventing the methane gas from reaching the

atmosphere.

The objective of this paper is to characterise methane hydrate soils from a geotechnical point of view.

Unfortunately, obtaining natural methane hydrate soil cores in intact state is difficult as the methane

hydrate can easily dissociate during coring and sampling (i.e. reduction in pressure and increase in

temperature). In this regard, special sampling devices to maintain the in situ pressure and temperature have

been developed (e.g. Tréhu et al., 2003). At present, limited numbers of geotechnical data are available

for natural methane hydrate soils. In this paper, using the data available in literature, geotechnical

data obtained from four sites, where natural soil samples are recovered and tested, are synthesized and

compared. The four sites are (a) Nankai Trough, (b) Mallik-Mackenzie Delta (include Mallik 2L-38 and

Mallik 5L-38), (c) Blake Ridge and (d) Hydrate Ridge. Their locations are identified in Figure 5.

Typical soil profiles of the four sites are shown in Figure 6. Nankai Trough and Mallik-Mackenzie

Delta sites are composed of alternating layers of sandy and clayey layers, whereas Blake Ridge and

Hydrate Ridge sites are composed of more or less homogeneous silty clay deposit. The different soils

profiles are due to different depositional environments leading to different patterns of methane hydrate

formation in their soil pores. In Nankai Trough and Mallik-Mackenzie Delta, methane hydrates are found

in sandy layers and the hydrate saturation at both sites can be 50% to 90% of the pore space (Uchida &

Tsuji, 2004). In Blake Ridge and Hydrate Ridge, methane hydrates are found in silty clay deposits and

smaller hydrate saturations of less than 10% of the pore space are reported.

2 GENERAL

2.1 Molecular structure of gas hydrate

The molecular structure of gas hydrate is dependent on the type of 'guest' gas molecule present inside

a caged structure of water molecules. There are three types of gas hydrate structures: Structures

I, II and H. 1200 1100 1000 900 800 700 600 500 400 300 200 100 0

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e l) Iperk sequence: Ice-bonded sand with conglomerate, silt and clay layers; abundant detrital organic material in some zones Mackenzie Bay sequence: Sand and weakly cemented sandstone, minor silt/shale interbeds Kugmallit sequence: Interbedded weakly cemented sand/sandstone and silt/siltstone Base of ice bearing permafrost 950 945 940 935 930 925 920 915 910 905 900 895 890 885 Unit 1: Fine- to medium-grained sand interbedded with gravel which fines upward to fine-grained sand with gradational increase in silt content Unit 2: Interbedded, fining upward successions of matrixsupported gravel to pebbly sand and fine sand Unit 3: Weakly bioturbated, clayey silt interbedded with fissile coal and silty sand Dolomite cemented sandstone Mackenzie Bay-Kugmallit sequence boundary (b) (a) 250 200 150 100 50 0 D e p t h (m e t e r b e l o w s e a f l o o r , m b s f) Unit 1: Calcareous silt and clay with volcanic ash layers Unit 2: Calcareous silt and clay with volcanic ash and thin sand layers Unit 3: Weakly consolidated calcareous silt and clay with thick and frequent sand layers ~ 950 m water depth

Figure 6. Lithology of sites (a) Nankai Trough: BH1 in landward slope at 60 km off Omaezaki, Japan with BSR

estimated at 290 mbsf (Hiroki et al., 2004). (b) Mallik Mackenzie Delta 2L-38: Left of this figure is the zoom in

view at depth 886-952 mbsf (Dallimore et al., 1999, Jenner et al., 1999) (c) Site 994, Blake Ridge (Matsumoto et al.,

1996) (d) Site 1244C, Hydrate Ridge located on the eastern flank of Hydrate Ridge~3 km northeast of the southern

summit (Tan, 2004). Gray zones indicate methane hydrate

occurrence inferred or confirmed. (c) 700 600 500 400 300 200 100 0 Depth (meter below seafloor, mbsf) Unit 2: Abundant diatom sand and lower nannofossil contents (average 10%) than those in Unit 1. Upper part of Unit 2 contains foraminifer-rich winnowed layers Unit 3: Monotonous diatom-bearing nannofossil-rich clay and nannofossil-bearing clay and claystone with average CaCO₃ contents of 15 wt%. BSR ~ 2800 m water depth (d) 300 250 200 150 100 50 0 Depth (meter below seafloor, mbsf) Unit 1: Dark greenish gray clay with scatter thin layers silty clay and fine silt Unit 2: Dark greenish gray silt clay and more very fine sand interbedded than Unit 1 Unit 3: Hard, indurated, dark greenish gray silty clay and clayey silt with scattered glauconite sand layers BSR ~890 m water depth Unit 1: Foraminifer-bearing nannofossil-rich clay and nannofossil clay with carbonate content as great as 50 wt% CaCO₃

Figure 6. Continued

Figure 7. Configuration of methane gas molecule surrounded by water molecules - Structures I, II and H. The most common form is Structure I (see Fig. 7), which entails a centred cubic cage accommo

dating a methane molecule. The formulation for ideal composition of Structure I methane hydrate is

CH₄ · 5.75H₂O. Other possible gases that can be present in this structure are ethane, carbon dioxide and

hydrogen sulfide. Most of gas hydrates are registered as 'non-stoichiometric' hydrate and this causes

more water molecules to form than in ideal composition (Sloan, 1998). Structure II has a diamond cubic cage as shown in Figure 7 and can accommodate a larger guest

molecule, such as nitrogen, propane and iso-butane. The cavity in both Structure I and II usually can

only contain one guest molecule. Structure II hydrate is more thermodynamically stable than Structure I

hydrate because Structure II can accommodate a larger gas molecule, which will act as diluent in the fluid

phase. Having the diluent results in a lower pressure and higher temperature for stabilisation than fluid

composed of smaller molecules for hydrate formation (Sloan, 1998). Assuming the sources of Structure

I and II are sufficient, Structure II will have a thicker hydrate layer than Structure I. For this reason,

natural methane hydrate deposits with Structure II are likely to result in a production of more gas than

deposits with Structure I. However, it is important to note that Structure II has a higher water/gas ratio

than Structure I. That is, more gas will be produced from Structure I hydrate than Structure II hydrate

with the same mass. The third structure, Structure H, as shown in Figure 7, is the least common in nature and is present

as hexagonal crystallographic system (Sloan, 1998). This structure requires two sizes of molecules as

guest molecules to stabilise the structure. Generally, Structure I hydrate is the most common structure to be found and is well related to biogenic

gases, whereas Structures II and H are found in an environment rich of heavier hydrocarbon associated

with thermogenic gases. Methane hydrates at Nankai Trough, Mallik-Mackenzie Delta and Blake Ridge

sites fall in Structure I hydrate category (Lorenson et al., 1999; Paull et al., 2000; Waseda & Uchida,

2002) even though mixed biogenic and thermogenic gases are encountered in Mallik-Mackenzie Delta.

In Hydrate Ridge, Structure I hydrate is found at Sites 1244-1247 on the flank with low gas flux,

whereas small amount of Structure II hydrate is found at Sites 1248-1250 on the summit with high gas

flux accompanied by the presence of propane (Milkov et al., 2005).

2.2 Origin of methane

There are two types of methane sources available: biogenic and thermogenic gases. Biogenic gases are

produced as a direct consequence of bacterial activity and are usually generated few metres below the

seabed. In general, generation of biogenic gas is associated with fine-grained sediment due to its higher

initial organic content (Booth et al., 1996). Thermogenic hydrocarbon gases usually occur at sub-bottom depths exceeding 1000 m and are com

posed of more complex hydrocarbons formed at higher temperatures from fossil organic matter (i.e.

kerogen or oil) (Sassen & MacDonald, 1994). It is usually associated with oil and gas reservoirs, but

Hydrate Ridge site is one of the first evidences where no active petroleum system is thought to exist

(Milkov et al., 2005). Thermogenic gas may indicate considerable upward migration of gas if it is iden

tified in the hydrate system. This pathway is commonly aided by faults, joints and fractures system as a

fluid conduit. Hence, thermogenic hydrate is less distributed and more concentrated in comparison with

biogenic hydrate (Sloan, 1998). However, there is no clear-cut relationship that biogenic and thermogenic hydrates are formed at

shallow and deep depths, respectively. In Gulf of Mexico and Caspian Sea, thermogenic gases are

observed even though gas hydrate is found near to the seafloor. -80 -70 -60 -50 -40 -30 -20 Methane & 13 C(o /oo) 1 10 100

1000

10000

100000

C 1

$C_1 / (C_2 + C_3)$

$C_2 +$

C_3

) Biogenic Thermogenic Mixed Gulf of Mexico Offshore
Northern California Black Sea Okhotsk Sea Offshore Nigeria
Nankai Trough Blake Ridge Offshore Peru Offshore Oregon
Offshore Guatemala Mackenzie Delta North Slope Alaska
Caspian Sea

Figure 8. Boundary between biogenic, mixed and thermogenic gases by the relationship between $C_1 / (C_2 + C_3)$

ratios and $\delta^{13}C$ values of CH_4 (modified from Waseda & Uchida, 2002).

The methane source can also be classified based on carbon isotopic composition relative to the

Peedee Belemnite (PDB) standard ($\delta^{13}C$; the units of parts per thousand (‰)) and the ratio of

$[methane(C_1)]/[ethane(C_2) + propane(C_3)]$. As shown in Figure 8, carbon isotopic composition that

is less/lighter than 60‰ and $C_1 / (C_2 + C_3)$ exceeds 1000 is classified as biogenic gases, whereas carbon

isotopic composition that is more/heavier than 60‰ and $C_1 / (C_2 + C_3)$ less than 100 is categorised as

thermogenic gas (Bernard et al., 1976). Figure 8 shows more hydrate sites are associated to biogenic gas

than thermogenic gas.

2.3 Formation

The presence of gas hydrate is controlled by the requirement of high pressure, low temperature and

source availability to form hydrate: gas as the 'guest' molecule and fresh water as the 'host' molecules.

These conditions can be satisfied in continental margin and permafrost regions.

In continental margin, high sedimentation rates and high

organic matter contents set it as an ideal

place for hydrate nucleation. The continental margin sediments are enriched in organics owing to two

factors: (i) higher biological production in the overlying water nearer to continental margin because of an

increased input of nutrients from rivers or from coastal upwelling and (ii) more organic matter remained

in rapidly deposited sediments because there is less destruction of organic matter by organism living

at the sediment-water interface prior to burial. Sea level fluctuation will also affect the magnitude of

pressure exerted to the sediments. When the sea level is high, the temperature at sea bottom is low, which

favours the conditions for hydrate formation.

In abyssal, it is unlikely to form gas hydrate even though the pressure and temperature conditions are

satisfied. This is because few thousands of years are available for decomposition of organics by organisms

before burial and, as a result, the sediments are very low in organic matter in pelagic sediments (Berner,

1980). Hence, the requirement for high concentration of methane is not fulfilled.

In high latitude areas like in permafrost, hydrate-bearing zone is available at shallower depths com

pared to continental margin areas because of the constant low temperature of the permafrost base $\sim 0^{\circ}\text{C}$

(e.g. -1°C for Mallik-Mackenzie Delta; Majorowicz & Smith, 1999 and Prudhoe Bay Alaska; Collett

et al., 1983). This low temperature has suppressed the high pressure requirement and only relatively high

pressure is needed for hydrate formation - gas hydrate formation is more sensitive to temperature rather

than pressure at permafrost sites. Taking into account that

fresh water source is sufficient, minimum methane solubility is required to

nucleate gas hydrate. Methane solubility in water increases with decreasing in temperature and increasing

in pressure. With presence of gas hydrate, methane solubility reduces with decreasing temperature as

the gas in the solution is transferred into hydrate (Zatsepina & Buffett, 1997; Max & Holman, 2003).

Hence, methane solubility is more influenced by temperature rather than pressure and low temperature

favours gas hydrate formation.

3 ENGINEERING GEOLOGY

3.1 Depositional environment

As discussed in the previous section, methane hydrate formation can take place in a wide range of

depositional environment such as in subduction zone, marine and permafrost deposits. Four sites are

identified in this study as examples of sites where methane hydrate soils are found (see Figure 5 for the

locations). The details of each site are given below.

(a) Nankai Trough (Fig. 6(a)) Nankai Trough lies in a subduction zone between the Eurasian and Philippine Sea plate, which extends about 700 km SSW-NNE offshore SE Japan at a rate of 2-4 cm/yr. The age of the sediments is between Quaternary to Pliocene (Matsumoto, 2000). The deposits largely consisted of interlayers of siliclastic hemipelagic clayey sediments with abundant sandstone turbidites. The abundance of sandy turbidite layers increases with depth and the thickness of a turbidite layer ranges from a few tens of centimetres to one metre. Methane hydrate is found in two zones: 40-130 mbsf and below 195 mbsf as shown in Figure 6(a). A total gas hydrate thickness of approximately 20 to 24 m has been identified (Uchida et al., 2004b). Methane hydrate is found in the sandy turbidite layers and the hydrate saturation goes up to 90% (Uchida & Tsuji, 2004). A microscope photograph of a methane hydrate soil taken from the site is shown in Figure 9(i).

(b) Mallik-Mackenzie Delta (Fig. 6(b)) Mallik-Mackenzie Delta site is located at the north-west corner of Canada. The nature of deltaic deposits is associated with changes of channel migration and variability in surface topography with time. This depositional environment resulted in layering of sediments with a complicated subsurface structure as shown in Figure 6(b). Gas hydrate is predominantly found in the lower Mackenzie Bay and upper Kugmallit Sequence at 897 to 922 m with typically 50 to 90% pore saturation (Uchida & Tsuji, 2004). The hydrate bearing layer consists of coarse-grained sand with thin beds of sandy conglomerate deltaic deposits (Jenner et al., 1999). As indicated in Figure 9 (ii), some tubes structures found indicating possible gas pathway for hydrate formation (Techmer et al., 2005).

(c) Blake Ridge (Fig. 6(c)) Blake Ridge is located at the eastcoast of the US. The sediments were deposited by the south-flowing Western Boundary Undercurrent that sweeps southward along the Atlantic Margin (Paull et al., 2000). The soil is characterised by continental contourite deposits of Tertiary to Quaternary homogenous nannofossil-rich clays in both lateral and vertical directions as shown in Figure 6(c). Gas hydrate is concentrated at two depth zones: between 185 and 260 mbsf and between 380 and 450 mbsf. The existence of hydrate at the lower zone can be explained by supersaturated methane gas, while that at the upper zone is due to the changes in lithology where the increase in siliceous microfossils increases the size of pore space. Despite the homogenous nannofossil clay, both hydrate zones are highly heterogenous; nodules, fracture fillings and thin layers embedded in clay with sharp hydrate are found. The largest piece of gas hydrate sample was more than 30 cm discovered at Site 997 as indicated in Figure 9 (iii) (Paull et al., 1996).

(d) Hydrate Ridge (Fig. 6(d)) Hydrate Ridge is located at the westcoast of the US in the Cascadia accretionary complex formed as the Juan de Fuca plate subducts obliquely beneath North America at a rate of 4.5 cm/yr. It covers the area of 25 km long and 15 km wide and large volumes of sandy and silty turbidites exist on the subduction plate (Mackay, 1995). The age of the sediments is between late Pliocene to late Pleistocene. The lithology of the site is shown in Figure 6(d). The boundary between Units I and II is determined by the decrease in the presence of turbidites in the younger lithostratigraphic unit and the boundary between Units II and III was interpreted as highly deformed sediments of accretionary complex. The changes from

fractured claystone interbedded with glauconite-bearing to glauconitic silts and sands were noted below 245 mbsf (Unit III) (Tréhu et al., 2003). Existence of hydrates was interpreted at 30-125 mbsf as several centimetre-thick veins of gas hydrate were discovered.

(iii) Blake Ridge: Massive piece of gas hydrate at 331 mbsf, Site 997 with ~5 cm × 14 cm in size, white

coloured, and coated by greenish gray drilling slurry with bubbles within the slurry (after Paull et al.,

2000) (i) Nankai Trough: Gas hydrate samples recovered with whitish colour from gas hydrate-bearing sand at 260.63 mbsf in Hole PSW-2 (after Uchida & Tsuji, 2004) (ii) Mallik 5L-38: (a) Gas hydrate with tubes indicating possible pathway for gas migration with zoom view in (b) (after Techmer et al., 2005) (a) (b)

Figure 9. Microscopic features of natural gas hydrate in sandy hydrate sites (i) Nankai Trough (ii) Mallik 5L-28

and macroscopic features in clayey hydrate site (iii) Blake Ridge.

Table 1 lists the sedimentation rates of the four sites. Nankai Trough site has a high sedimentation rate

compared with the others due to the overall thickness of turbidites that increases from about 100 m to

430 m (Shipboard Scientific Party, 2001). If sedimentation occurs very rapidly with little interruption,

steady thermal state cannot be achieved at the shallow sedimentary formation. Consequently, the conduct

ive heat flow measured at a shallow sub-bottom depth can be significantly less than the actual geothermal

heat flow entering from the bottom of the formation. Hence more gas hydrate can be formed. Kvenvolden

& McDonald (1985) suggests sedimentation rates between 30 m/m.y. and 300 m/m.y. are necessary for Table 1. Sedimentation rates at four hydrate sites. Sedimentation rate Site* (m/m.y.) Reference Nankai Trough 300-1340 Sreaton et al. (2001) Mallik-Mackenzie Delta N.A. - Blake Ridge 350 Paull et al. (2000) Hydrate Ridge 100-630 Tan (2004) *N.A. = Not Available.

hydrate formation, whereas Finley & Krason (1986) set these limits to between 5 and 1000 m/m.y. based

upon evidence from Deep Sea Drilling Project (DSDP) Legs 67 and 84 (Sloan, 1998). Depositional environment will yield different physical characteristics especially the homogeneity

of gas hydrate sediment. Passive region like Blake Ridge consists of more homogenous sediments

when compared with accretionary active region such as Nankai Trough and Hydrate Ridge. These sites

(Nankai Trough and Hydrate Ridge) have sediments with more fractured and sheared features due to the

compressive force by the tectonic movement (Ciesnik & Krason, 1989). The occurrence of gas hydrate at

shallow depths in the active convergent regions is attributed to the active gas venting and fluid seeps. This

can be seen in Nankai Trough and Hydrate Ridge where gas hydrate is found near the surface of seabed. Although sedimentation rates and depositional environment influence the development of methane

hydrate soil, the amount of methane hydrate in the soil pores is more controlled by the type of sediment

present. For example, although Nankai Trough and Hydrate Ridge is in accretionary active region, the

amount of methane hydrate in Nankai Trough is closer to Mallik-Mackenzie Delta which is in terrestrial

deposition. NankaiTrough and Mallik-Mackenzie Delta sites are composed of alternating layers of sandy

and clayey layers. At both sites, methane hydrates are found in sand bearing deposits and more methane

hydrates are found here compared to Hydrate Ridge and Blake Ridge, in which clay sediments dominate.

3.2 Gas hydrate growth in soil pores

Water is one of the input sources for gas hydrate

formation. Different initial water saturations in the soil

pores may yield different gas hydrate morphologies. One molecule of methane has to be accompanied by

5.75 water molecules in Structure I. Thus, a lower water content in surrounding gas hydrate bearing sedi

ment than in non-hydrate sediment will be resulted. The “mousse-like” texture of gas hydrate sediment

(like Fig. 9(iii)) seen during the extraction of gas hydrate samples (i.e. Blake Ridge; ODP Leg 146, Site

892A Paull & Ussler III, 2000) is the reverse process of gas hydrate formation, indicating an increase

in water content near the surrounding sediment and/or to be associated with gas expansion during core

recovery. For water saturated sediment (free water environment where gas controls hydrate concentration in pore

space), free water allows hydrate to form from bubbles within the pore space. This may be encountered

in active margin region like accretionary margin (Cascadia Margin, Gulf of Mexico; Priest et al., 2005)

or at the base of Bottom Simulating Reflector (BSR) (discussed later) where free gas are commonly

encountered. For partially saturated sediment (free gas environment where water controls amount of hydrate in

pore space), the free gas leads to forced hydrate growth at grain contacts (Kingston et al., 2006). This

condition is found when the excess methane gas is dissolved in the water. An increase in initial water

content of the soil can result in an increase in hydrate saturation as shown in Figure 10. The data shown

are from synthetic methane hydrate soils, which were created using different initial water contents. At

high initial water contents, however, gas hydrate saturation dropped with increase in initial water content.

This is because the gas hydrate formed around the gas injection point plugged the soil pores and it became

difficult for the gas to penetrate into the other parts of the specimen. Silica sand and sandstone are typical components of sediments, in which natural gas hydrates are

observed to form. These minerals (mainly silicate) are not absorbent of large amounts of water and

hence gas hydrates would form from water existing originally in the spaces between the particles (Uchida

et al., 2004a). Gas hydrate soils can be categorised into four growth patterns as indicated in Figure 11

(Helgerud et al., 1999; Ecker, 2001; Clayton et al., 2005; Kleinberg & Dai, 2005):

(a) Model A: hydrate is part of the pore fluid: pore filling. Gas hydrate is suspended in the pore fluid and only affects the bulk modulus of pore fluid and bulk density of the sediment. H y d r a t e s a t u r a t i o n (%) Initial water content (%) 0 20 40 60 80 100 0 20 40 60 80

Figure 10. Relationship between initial water saturation and hydrate saturation (Ebinuma et al., 2003).

Figure 11. Different morphology of gas hydrate (A) pore filling (B) supporting matrix (C) cementation (D) coating

(modified from Kingston et al., 2006).

(b) Model B: hydrate is part of the dry frame: supporting matrix. Sediment porosity reduces with gas

hydrate saturation. Dai et al. (2004) describes that this supporting matrix model matches the gas

hydrate occurrence in Mallik 5L-38.

(c) Model C: hydrate is part of the dry frame: cement grain contacts. The presence of gas hydrate also

reduces the sediment porosity. This type is only valid for porosities less than 40% and cementation

growth of hydrate is unlikely at Blake Ridge as the

sediments in this region are highly unconsolidated

(Ecker, 2001).

(d) Model D: hydrate is part of the dry frame: grain coatings. Similar to Model B and C, sediment

porosity reduces with increase in hydrate saturation.

However, there are two major growth patterns which are Models A and C. Nankai Trough, Mallik

Mackenzie Delta, Blake Ridge and Hydrate Ridge appear to have a pore filling type of hydrate (Model

A) (Matsumoto, 2000; Dallimore & Collett, 2005; Paull et al., 2000). The importance of the gas hydrate

growth pattern is significant because it affects the mechanical properties of gas hydrate, which will be

discussed further in the engineering properties section.

Pore filling gas hydrate (Model A) can be created synthetically by growing methane hydrate in a

mixture of sand and fine ice particles below ice melting point. This process forms hydrate mainly inside

the soil pores and weaker bonds are created. Soil samples with different methane hydrate saturations can

be created by changing the induction period (Masui et al., 2005a).

Figure 12. Macroscopic of gas hydrate (A) Disseminated (B) Nodular (C) Massive (D) Vein (E) Veinlets (F) Layer

(after Tréhu et al., 2003). Cemented gas hydrate (Model C) can be created synthetically by pumping methane gas into wet

sand. Initially mixture of water and sand grain is compacted with controlled water content. The water

accumulates at particle contacts due to capillarity. After setting the sample into a core holder, methane

gas is introduced by diffusion and remaining air is displaced by methane gas. Methane gas is then

injected into the core at high pressure and the temperature is reduced to form gas hydrate. This process

forms hydrates at particles contacts and strong bonds are produced. Soil samples with different methane

hydrate saturations can be created by changing the initial water content. The data presented in Figure 10

are produced from this preparation method. In reality, the macrofabric of natural gas hydrate soils is more complex than the models described

above due to macroand micro-heterogeneity of field soils as shown in Figure 12. The macrofabric

of hydrate is divided into six categories: (1) pore space hydrate, (2) platy hydrate, (3) layered/massive

hydrate, (4) disseminated hydrates, (5) nodule hydrate and (6) vein/dyke hydrate (Uchida et al., 2000).

The observed macrofabrics are believed to be linked to the mechanisms of hydrate growth; Malone (1985)

postulate that the initial formation of disseminated hydrate crystals grows to nodules, then to layers and

Table 2. Summary of known gas hydrates ever recovered world wide showing the mode of occurrence

(Uchida, 1999). Water depth Sub-bottom depth

Area (m) (m) Occurrence Gas hydrate

Onshore

Mackenzie-Delta 1998 - 900+ pore space, vein up to 12 mm

Mackenzie-Delta 1992 - 336, 354 platy, pore space, vein up to 50 mm thick

Offshore

Nankai Trough 945 195-268 pore space <1 mm

Okhotsk Sea 1991 710 1 layered, nodule up to 10 mm thick

Offshore Oregon 1996 0.5 layered up to 10 mm thick

Offshore Oregon

Leg 146, Site 892 670 3.6 disseminated 4-5 mm

Offshore Guatemala

Leg 67, Site 497 2347 368 disseminated 2 cm

Leg 67, Site 498 5478 310 disseminated

Leg 84, Site 568 2010 404 nodule 10.4 cm 3

Leg 84, Site 570 1698 249 nodule, massive 1.05 m * 6 cm phi

Offshore Costa Rica

Leg 84, Site 565 3099 285, 319 disseminated 1.5 cm 3

Offshore Peru

Leg 112, Site 685 5070 99, 116 disseminated 1.18 cm 3

Leg 112, Site 688 3820 141 disseminated, vein a few mm

Blake Outer Ridge

Leg 76, Site 533 3184 238 nodule 4 cm phi Leg 164,

Site 997 2800 330 nodule, vein up to 35 cm

Okushiri Ridge

Leg 127, Site 796 2570 90 disseminated 5 cm 3

Gulf of Mexico

Leg 96, Site 618 2400 27 disseminated 1-4 mm

Garden Banks-338 850 2.8 disseminated 20 mm

Green Canyon-257 880 4.2 disseminated 10*3 mm

Green Canyon-234 590 1.2, 2, 8 nodule >150 mm

Mississippi Canyon 1300 3.8 disseminated 2 mm

finally to a massive hydrate (Sloan, 1998). Another hypothesis is that massive hydrates (nodular, layered

and massive) might result from episodic melting-recrystallization (and related temperature) fluctuations

arising from sea level changes (Sloan, 1998).

Table 2 shows the types of gas hydrate occurrence at different sites with water depth ranges from

590-5500 m with sub-bottom depth down to 400 m for offshore sites and 900+ m for onshore sites.

Most of the gas hydrate found is the disseminated form as shown in Figure 12 (A). The thickness

of gas hydrate varies irrespective of the type of hydrate growth. Further information can be found in

Booth et al. (1996).

3.3 Post depositional environment

Sediment erosion influences the stability of methane hydrates in the soils (Krasen & Ridley, 1985). The

initial effect is the local increase in heat flow as the sediment readjusts to the new thermal regime. As

long as the sediment is under hydrostatic conditions, erosion will have no effect on the pressure regime

because the total pressure is usually governed by the water pressure under the deep sea conditions.

However, if methane hydrate growth is promoted by geostatic pressure, then erosion will cause a slight

depressurisation of the sediment column. The ultimate effect is that the base of the hydrate zone will

move upward or downward to establish a new equilibrium position determined either by pressure or

temperature.

3.4 Bottom Simulating Reflector

Bottom Simulating Reflector (BSR) is considered as the most important tool to detect gas hydrate

formation using seismic methods. BSR marks the base of the gas hydrate stability zone, which is

the intersection between geothermal gradient and phase stability boundary. It detects the interface by

identifying the P-wave velocity difference in the two layers since the upper gas hydrate layer has a higher

P-wave velocity while the lower free gas layer has a lower P-wave velocity. Site 997 in Blake Ridge is

one of the examples where the BSR coincides with the gas hydrate layer as shown in Figure 13(a). BSR occasionally fails to recognise the presence of gas hydrate. Ocean Drilling Programme (ODP)

Leg 84 offshore Guatemala (the first and thickest gas hydrate marine ever recovered), Site 994 in Blake

Ridge and Gulf of Mexico are the examples of gas hydrate occurrence without BSR (see Site 994 in

Fig. 13(a)). The presence of gas hydrate without BSR may be attributed to the lack of free gas at the

bottom or geological feature constraint. The existence of BSR does not necessary indicates the presence of gas hydrate. The existence of BSR

without gas hydrate may indicate diagenesis silica effect from opal-A to opal-CT or opal-CT to quartz

(high water content and low bulk density) (Volpi et al., 2003). Double BSRs are recognised in Kodaiba knoll in the easternmost Nankai Trough due to sea bottom

warming and tectonic uplift (see Fig. 13(b)) where hydrates are found (Foucher et al., 2002).

Figure 13. Examples of BSR from seismic reflection profiles in (a) Blake Ridge: absence of BSR at Site 994 (Paull

et al., 2000); (b) Upper slope of eastern Nankai margin METI: double BSRs observed (Foucher et al., 2002).

In permafrost regions, BSR is not commonly used due to similarity of P-wave velocity for hydrate

soil and ice. Thus, BSR cannot be a sole indicator of the

presence of gas hydrate. Other important

indications of hydrate presence include pore water salinity and temperature anomalies. Effect of pore

water salinity will be explained more in the pore water chemistry section. The dissociation of gas hydrate

can be detected by the cooling effect due to its endothermic process. For example, gas hydrate samples

that are recovered at approximately 900 m from Mallik 2L-38 have temperature around 7 °C, but some

ice was formed around the samples. Mimicking the ground freezing application to obtain high quality

sample, less disturbed sampling can be achieved if this process occurs (Winters et al., 1999a). It also

acts as self-preservation, preventing gas hydrate from dissociating further.

4 COMPOSITION

4.1 Particle size distribution

To date, more cases of gas hydrate occurrence are reported to be associated with coarse-grained sediment

(sand) than fine-grained sediment (clay). In fine-grained sediment, the gas and liquid transport rates

may be very slow, which limits the extent of hydrate accumulations. In coarse-grained sediment, there

are more opportunities for methane gas and water to interact (Clenell et al., 2000; Milkov & Sassen,

2002). It is important to note that some textures like breccias may be a consequence of expansion of the

pore fluid in veins or joints as it converts to gas hydrate (Black Sea, Caspian Sea; Booth et al., 1996).

Carbonate gravel that may be a product of the dissociation of hydrates (Hydrate Ridge; Bohrmann et al.,

2002) could mask the true probability of lithology of natural gas hydrate occurrence.

Figure 14 shows the particle size distributions of the hydrate-bearing-sand at Nankai Trough and

Mallik-Mackenzie Delta and of the hydrate-bearing-clay at Blake Ridge and Hydrate Ridge and Gulf of

Mexico. The mean particle size is about 0.1-0.3 mm for the Nankai Trough and Mallik samples, whereas

that is about 2-5 μm for the Blake Ridge and Hydrate Ridge samples. However, in Nankai Trough and

Mallik-Mackenzie Delta, if the whole soil profile section is considered, the soil profiles consist of layers

of sandy and clayey soils and the particle size distributions vary dramatically as shown in Figure 15. It

shows that Mallik-Mackenzie Delta soils have greater variety of soil types compared to Nankai Trough

soils. This is attributed to the channel migration in the fluviodeltaic environment as discussed earlier.

Unlike Nankai Trough, the sandstone gravel in Mallik-Mackenzie Delta is not always saturated by gas

hydrate (Matsumoto, 2002).

Particle size distribution of the sediment and layering can control the spatial distribution of methane

hydrate. Methane hydrate nucleation is initially promoted in coarser-grained parts of the sediment and

growth may then spread into the finer-grained sediment. In coarse sediment, methane hydrates are often

Figure 14. Particle size distributions of methane hydrate soils in sandy soil (a) Nankai Trough (Masui et al., 2006)

(b) Mallik 5L-38 (Jenner et al., 1999) (c) Blake Ridge (Paull et al., 2000) (d) Hydrate Ridge (Tan, 2004).

Figure 15. Particle size distributions of Nankai Trough, NK (Masui et al., 2006) and Mallik-Mackenzie Delta,

MMD (Jenner et al., 1999).

Table 3. Variation of total organic content (TOC). Average TOC value

Site (%) Reference

Nankai Trough 0.5-0.8 Moore et al. (2001)

Mallik 5L-38 0.2-3.1 Waseda & Uchida (2005)

Blake ridge 0.5-1.5 Paull et al. (2000)

Hydrate ridge 1.1-1.7 Shipboard Scientific Party (2003)

found to cement the grains throughout the matrix, whereas in finer sediment they frequently occur as

discrete nodules, sheets or lenses. The depression of water vapour pressure is inversely related to the

capillary radius. The capillary force in fined-grained sediment reduces the freezing point of water and

shifts the location of BSR. Hence, additional pressure is required for hydrate equilibrium. This is also

one of the reasons why BSR is discontinuous at certain locations of hydrate sites (Clennell et al., 2000). Considering the production aspect, fine-grained sediment environment makes potential recovery time

long and expensive. Therefore, extracting gas from hydrate located in coarser-grained host sediment

reservoirs may be more economically feasible.

4.2 Organic content

Table 3 lists the total organic contents of samples from the four sites. The minimum total organic carbon

(TOC) for methane hydrate soil is often quoted as 0.5% to sustain bacterial methanogenesis. If the site

has less than this amount, there is often some additional assistance like fault acting as a pathway for

deeper methane gas source to transfer to the shallower gas hydrate stability zone, such as in Nankai

Trough. It is believed that the source of gases in Nankai

Trough is considered to be accumulated by

recycling process since early Tertiary (Matsumoto, 2002).
Mallik 5L-38 exhibits TOC values between

0.2 to 3.1%, which are considered to be representative at
this site. On the other hand, the TOC values

at Mallik 2L-38 were up to 13.1%. This high value is
attributed to mud additives contamination from

the borehole and was interpreted as lechitin, which has
high hydrogen/carbon ratio (Waseda & Uchida,

2005, Uchida et al., 1999). In clay sediment hosted gas
hydrate sites, Blake Ridge is a passive margin setting with
high organic

content and low fluid flow (0.2 mm/yr), yielding hydrate
saturation of 4-8%. In contrast, Cascadia

Margin is located in a subduction region with low organic
content and high fluid flows (1-2 mm/yr),

yielding hydrate saturation of 20-30% (Davie & Buffett,
2003). This implies that, once the minimum

requirement of total organic content is exceeded, fluid
flow rate also plays an important role in determin

ing gas hydrate saturation. Therefore, accretionary prism
is an ideal place for gas hydrate exploration

due to high fluid flow.

4.3 Carbonate content

Carbon element is released upon methane hydrate
dissociation, resulting in carbonate formation. The

driving force for carbonate formation is the increase in
alkalinity from anaerobic methane oxidation via

sulfate reaction. This situation can be encountered in
Hydrate Ridge (Bohrmann et al., 2002; similar as in

Gulf of Mexico, Francisca et al., 2005). The carbonate
content varies between 1.31-10.72% where low

and high-magnesium calcites are the dominant carbonates

mineralogy (Teichert & Bohrmann, 2006). In

Blake Ridge, carbonate carbon is in the range of 0.89-3.08% (Paull et al., 2000). In Mallik 5L-38, the

carbonate content varies between 0.83-20.83% with high carbonate content in lignite lithology (Haberer

et al., 2005). Information about carbonate content in Nankai Trough is not available.

4.4 Pore water chemistry

The stability of methane hydrate is affected by the presence of salt as it reduces the freezing point of

ice and seawater pressure. The concept is similar to salt application for ice melting on the road during

winter. Gas hydrate formation excludes ions of dissolved salt from seawater and, as the gas hydrate

formation proceeds, salinity in the pore water increases if there is no supply of fresh water. This results

in the reduction of gas hydrate formation rate. In Mallik 5L-38, the pore water salinities is 45 ppt (parts

per thousand), which is higher than the normal seawater salinity (35 ppt) (Dallimore & Collett, 2005).

This represents an upward shift of the gas hydrate stability field of approximately 110 m from the depth

expected for gas hydrate.

Since gas hydrate excludes all ions upon formation, the increased chlorinity in water reflects the

amount of pore water that accompanies gas hydrate in the measurements. That is, increased patterns

of chlorinity with depth indicate gas hydrate formation. Hence, pore water refreshing is an important

indicator for gas hydrate dissociation.

In some cases, degree of chlorinity depends whether the hydrate system is open or closed to ambient

waters (Matsumoto, 2002). With the open system, the saline water surrounding the hydrate can mix with

the ambient water and chloride concentration of residual water should be close to common seawater

chloride concentration (550 mM). However, not all the increased patterns of chlorinity with depth are

attributed to gas hydrate formation. Possible other attributions may include mineral dehydration upon

increasing pressure, membrane filtration, dewatering of subducting sediment, input of meteoric waters

and deposition of brackish near coastal sediment 'slab' into the deep sea (De Lange & Brumsack, 1998).

The increased of chlorinity in closed system retards the gas hydrate accumulation process.

For hydrate bearing clay sediment, an increase in electrical double layer repulsion induced by fresher

water after gas hydrate dissociation will change the clay structure from face-to-face structure before gas

hydrate dissociation to dispersed structure after dissociation (Francisca et al., 2005).

Oxygen isotope anomaly is also an indication of gas hydrate formation as gas hydrate concentrates

isotopically in heavier oxygen ($\delta^{18}O$) in its cage structures (Hesse & Harrison, 1981; Davidson et al.,

1983). Isotopically heavy oxygen containing siderites in Blake Ridge sediments has been documented by

Matsumoto (2000). The gas hydrate saturation can be determined by chloride and oxygen isotope anomaly

lies. However, the discrepancy between chloride and oxygen anomalies in predicting gas hydrate saturation

may be attributed to the uncertainties in determining the baseline profiles for the pristine interstitial

waters and partially due to filtration/adsorption effects during water extraction (Matsumoto, 2000).

5 STATE AND INDEX PROPERTIES

5.1 Density, porosity and water content

The bulk densities of the gas hydrate bearing sandy sediments at depths from 204 to 268 mbsf (1149 to

1213 mbsl) in Nankai Trough are approximately 2.35 g/cm³ (Uchida & Tsuji, 2004). In Mallik-Mackenzie

Delta, the bulk densities are around 1.90–2.10 g/cm³ at the gas hydrate sand bearing interval depth of

897–922 m (Winters et al., 1999b). The bulk densities of Blake Ridge hydrate soil are about 1.10 to

1.90 g/cm³ (Collett, 2000). In Hydrate Ridge, the density of the sediments at top 50 mbsf increases more

or less linearly with depth at 1.20 to 1.60 g/cm³ and below 50 mbsf, the densities are more uniform

around 1.60–1.80 g/cm³ (Shipboard Scientific Party, 2003). The bulk density of pure methane hydrate is 0.90 g/cm³. In Hydrate Ridge, it is reported that the bulk

densities of the natural gas hydrates are around 0.40–0.70 g/cm³, which is significantly less than the

theoretical value. This is due to the pores in situ that are filled with gas rather than water. The implication

of this low bulk density is the buoyancy effect that drives gas hydrate to float up. Some of gas hydrates

remain on the seafloor only because of the shear strength of the host sediment (Suess et al., 2002). Porosity and pore size distribution of sediments can be determined by using mercury porosimetry; a

conventional method used in petroleum industry to obtain absolute porosity of a porous substance. Other

methods include porosity derived from bulk density. In clayey sediments, errors can arise in porosity

measurement due to rebound effect or smectite dehydration during oven drying of samples (Screaton

et al., 2001). In the field, porosity is determined from Azimuthal Density Neutron (ADN) from logging

while drilling method. However, if the borehole is degraded, the value will be affected (Lee, 2000). In Nankai Trough, the porosity of the gas hydrate sand bearing sediment is approximately 40%,

whereas the porosities of the silt and muddy sediment are 27-50% (Uchida & Tsuji, 2004). In Mallik

5L-38, the sand layer (892-930 m) has generally uniform porosity (32-38%), whereas the silt rich

intervals generally have more variable porosities (30-40%). The gas hydrate sand-bearing sediment has

lower porosity than the upper and lower silt rich layers. This indicates that porosity is not the sole factor

that controls the distribution of gas hydrate (Winters et al., 1999b); large permeability, which is related

to particle or pore size, is important for gas to transport in forming hydrate. In Blake Ridge, the average

porosities of the hydrated intervals are 60-70%, which are derived from density logging (Lee, 2000).

Similar porosity values are reported for Hydrate Ridge (Tan, 2004). In Mallik 5L-38, water content is around 17-20% for the sandy sediment and 25-42% for the organic

layers. The organic layers have its capability of trapping moisture within the organic fibres (Winters

et al., 2005). Soil with high organic content and presence of permafrost can increase the water content of

the sediment (Winters et al., 1999b). Increase in density and reduction in water content with depth is the

normal trend for consolidation process. However, thawing of permafrost may result in underconsolidated

sediment if permafrost is formed before primary consolidation is completed (Winters et al., 1999b). In Blake Ridge, water content is 40-70% to a depth of around 700 mbsf and linearly reduces with depth

(Site 995; Winters, 2000b). The water contents of Hydrate Ridge soil are irregular with approximately

50-70%, but a trend of water content reducing with depth is observed (Tan, 2004).

5.2 Atterberg limit

The classification of the methane hydrate clays from Blake Ridge, Hydrate Ridge and Gulf of Mexico

are shown in Figure 16. (no information is available on Nankai Trough and Mallik Mackenzie Delta clay Limit limit (LL) 0 0 10 20 30 40 50 60 70 80 90 100 110 10 20 30 40 50 60 70 P l a s t i c l i m i t (P I) Blake Ridge (Paull et al., 2000) Hydrate Ridge (Tan, 2004) Gulf of Mexico (Francisca et al., 2005) Line A, PI = 0.73(LL-20)

Figure 16. Relationship between plasticity index and liquid limit.

data). Most of the clay hydrate sediments are in the region of inorganic clay of high to very high plasticity

according to BS5930 (1999). All the clay sediments are categorised in illite and kaolinite groups for

Blake Ridge soil (Paull et al., 2000) and illite group for Hydrate Ridge soil (Activity~1; Tan, 2004) and

Gulf of Mexico soil (specific surface area 57-88 g/m² ; Francisca et al., 2005).

Chuvilin et al. (2002) state that, although pure montmorillonite clay has higher initial water content

(70-13%) compared to kaolinite (44%) (i.e. more water source available for hydrate formation), kaolinite

sample can generated 44% gas hydrate saturation compared to montmorillonite sample (<22%). This

can be accounted by the fact that montmorillonite has malleable crystalline lattice and high specific

active surface (500 m² /g to <30 m² /g in kaolinite). As a result, pore water in kaolinite sample is less

attracted by the energy of mineral surfaces and thus provides a favourable condition for transformation

of water into hydrate (Chuvilin et al., 2002).

The presence of methane hydrate has little effect on Atterberg limits and the soil behaves like other

common engineering soils when it is disturbed. However, if there is a presence of high biogenic material,

it may cause the gas hydrate sediment to be poorly correlated with standard empirical relationships (like

peaty soils).

6 ENGINEERING PROPERTIES

As described above, methane hydrates are formed in the sandy layers of Nankai Trough and Mallik sites,

whereas they are formed in the silty clay sediments of Blake Ridge and Hydrate Ridge sites. For brevity,

the former sediments are called as 'sandy methane hydrate soils' and the latter as 'clayey methane hydrate

soils'. In this section, the engineering properties of these sediments are reviewed separately. At Nankai

Trough and Mallik sites, clayey sediments similar to those found at Blake Ridge and Hydrate Ridge sites

are found (Fig. 15), but they do not contain any methane hydrate. We will call them 'clayey no-hydrate

soils' and their properties are compared with 'clayey methane hydrate soils'.

6.1 Permeability

6.1.1 Sandy methane hydrate soils

For Mallik 5L-38 samples, the permeability is reported from 10^{-10} m/s in the gas-hydrate bearing-sand

and more than 10^{-5} m/s in the water bearing sand section (free gas-hydrate-bearing); a difference of

five order magnitudes (Dallimore & Collett, 2005). In Nankai Trough, when methane hydrate is not

present, typical permeability values of the sandy sediment are 10^{-5} – 10^{-9} m/s. This large variation in

permeability is due to the heterogeneity of the sandy layers at this site as discussed before.

Permeability reduces when there is a presence of gas hydrate, since some of the pore space that is

originally occupied by pore fluid will be replaced by gas hydrate and subsequently blocks the water

flow. Hence, gas hydrate saturation is important in determining the sediment permeability. As shown in

Figure 17, permeability decreases with hydrate saturation and the decrease is more significant at low

hydrate saturations (Nimblett & Ruppel, 2003). Synthetic hydrate soil made by gas diffusion method to

mimic the cemented gas hydrate (Model C in Fig. 11) also exhibits similar behaviour (Minagawa et al.,

2004). Similar pattern is also reported by Berge et al. (1999) in synthetic gas hydrate samples using

refrigerant trichlorofluoromethane (R11) as hydrate former.

Porosity can be divided into two categories (Aharonov et al., 1997): storage porosity (storage of gas

hydrate) and connecting porosity (path for fluid migration), where both type of porosities can influence

the gas hydrate saturation (Katsube et al., 2005). That is, porosity is not the only factor influencing the

overall permeability of a hydrate bearing soil layer. It is also associated with the thickness of the layer

and position of the observed base of gas hydrate zone in natural sediments. As hydrate accumulates,

particularly near the top of the free gas zone, pore space becomes clogged, which results in reduction

of methane flux from below. While the position at the top of the gas hydrate zone is largely determined

by the solubility of methane in seawater (Zatsepina & Buffett, 1997), the position of the base of gas

hydrate zone is believed to be controlled by the degree of permeability reduction due to hydrate formation

(Ruppel, 1997; Milkov & Sassen, 2002). The permeability reduction by restricting migration pathways

without a decrease in porosity is shown in Figure 18 at the interval 940-945 m of Mallik 5L-38.

6.1.2 Clayey methane hydrate soils

Hydraulic conductivity data of Hydrate Ridge soil was obtained from Constant Strain Rate Consolidation

(CSRC) oedometer tests. As shown in Figure 19, the permeability of Hydrate Ridge samples decreases
10.90.80.70.60.50.40.3 (1-S MH) 1E-010 1E-009 1E-008
1E-007 1E-006 1E-005 1E-004 1E-003 P e r m e a b i l i t y
(m / s) Mallik 2L-38 sand Silica sand No. 8 Silica sand
No.7 Toyoura sand 0.7 0.5 0.4 0.3 0.2 0.1 0.0 Gas hydrate
saturation, S MH 0.6

Figure 17. Measurement of water permeability under the presence of methane hydrate with natural sample from

Mallik 2L-38 and synthetic samples (silica and Toyoura sand) using gas diffusion method (Minagawa et al., 2005).

Figure 18. Effect of porosity and permeability for gas hydrate content in Mallik 5L-38. Effective porosity is meas

ured using helium porosity. MS-1 and MS-2 represent mudstone Units 1 and 2 respectively. 1 mD is equivalent to

1×10^{-8} m/s (Katsube et al., 2005).

with reduction in void ratio (Tan, 2004) and is in the range of 10^{-6} to 10^{-9} cm/s. There is no permeability

data available for clay sediments at the other sites.

6.2 Compressibility

6.2.1 Sandy methane hydrate soils

No data is available for the compressibility of the sandy materials at Nankai Trough and Mallik sites.

Figure 19. Hydraulic conductivity of Hydrate Ridge (Data from Tan, 2004). 1 10 100 1000 10000 Vertical effective stress (kPa) 0.4 0.6 0.8 1 1.2 1.4 1.6 1.8 V o i d r a t i o , e (b) Hydrate Ridge NCL

0.1 1 10 100 1000 10000 100000 Vertical effective stress (kPa)

0

0.5

1

1.5

2

2.5

3

V o

i d

r a

t i o

, e (a) Blake Ridge NCL

Figure 20. Constant strain rate consolidation results (a) Site 995, Blake Ridge (Data from Winters, 2000a) and

(b) Site 1244, Hydrate Ridge. Normal Compression Line (NCL) is indicated by dashed line (Data from Tan, 2004).

Table 4. Result of consolidation tests for clayey sediment. Pc Rate of sedimentation

Site Cc OCR (kPa) (m/m.y.) Reference

Blake 0.67-1.38 2.9 at 3.09 mbsf and 40-2730 350 Winters (2000a)

ridge <1 at deeper sediment

Hydrate 0.47-0.70 2.0-7.3 <33 mbsf and 40-400 100-630 Tan

(2004)

ridge <1 at deeper sediment

6.2.2 Clayey methane hydrate soils

CSRC oedometer tests were conducted on Blake Ridge and Hydrate Ridge samples. The results are

shown in Figure 20 and Table 4. The strain rate used in the CSRC oedometer tests was 0.5%/hour. The

compression index C_c of the soils was 0.97–1.38 and 0.47–0.70 for Blake Ridge soil and Hydrate Ridge

soil, respectively. No compressibility data on clayey sediments of Nankai Trough and Mallik-Mackenzie

Delta is publicly available.

Slow deposition in still water creates open random fabric with a high value of void ratio leading to a

larger compression index (Burland, 1990). Assuming minimal sample disturbance, Blake Ridge soil has OCR 700 600 0 1 2 3 4 5 6 7 9 500 400 300 200 100 0 D e p t h (m b s f) Blake Ridge (Winters, 2000a) Hydrate Ridge (Tan, 2004)

Figure 21. Variation of OCR with depth; Blake Ridge (Data from Winters, 2000a) and Hydrate Ridge (Data from

Tan, 2004). 1 10 100 1000 10000 Vertical effective stress (kPa) 0.4 0.6 0.8 1 1.2 1.4 1.6 1.8 V o i d r a t i o , e cemented NCL uncemented NCL

Figure 22. Influence of cementation in Hydrate Ridge. Normal Compression Line (NCL) is indicated by dashed

line (Data from Tan, 2004).

larger initial void ratios compared to Hydrate Ridge soil. The more open fabric of Blake Ridge soil may

be partly responsible for the large compression index. However, the Hydrate Ridge sample was highly

disturbed during recovery (Tan, 2004) and relatively rapid fluid flow in Hydrate Ridge may invalidate

the assumption made here. The overconsolidation ratio (OCR)

profiles of these two sites are shown in Figure 21. Both sites are

in heavily overconsolidated state at shallow depths. The reason for this is unclear but can be due to

removal of overburden by mass movement or cementation from organic matter. It is common to have a

high organic content at shallow depths especially in Hydrate Ridge. The presence of cementation will

increase the preconsolidation pressure compared to uncemented soil as shown in Figure 22.

0.1 1 10 100 1000 10000 Effective stress (kPa) 0.0 0.5 1.0 1.5 2.0 2.5 3.0 3.5 4.0 4.5 Void ratio, e Blake Ridge Hydrate Ridge w = 120 PI = 80 w = 80 PI = 50 w = 50 PI = 25 w = 30 PI = 12 Highly colloidal clays Colloidal clays Clays Silty clays Silts

Figure 23. Comparison of normal consolidation lines (NCLs) for clayey hydrate sediments. Plasticity index is

denoted PI and water content as w. The graph is redrawn from Mitchell & Soga (2005).

Compression test was also conducted on the horizontally cut specimen of Hydrate Ridge soil to

investigate the anisotropic behaviour. Results indicate that the horizontally cut sample has higher pre

consolidation pressure than the vertically cut sample, indicating K_0 greater than one. This also explains

the extremely high OCR values at the shallow depths in Hydrate Ridge (Tan, 2004).

In Blake Ridge, underconsolidated state is apparent at deeper depths; the porosity is relatively constant

with depth, which is in contrast with a traditional decreasing porosity trend with depth (Paull et al., 2000).

The rapid sedimentation rate that coincides with large excess pore water pressure indicates the completion

of primary consolidation process has not been achieved. Other possibility includes the dissociation of gas

hydrate that increases the pore water pressure in the hydrate layer (200–450 m). However, this possibility

is considered to be low because the non-hydrate layer also exhibits the same characteristic and there is no

visual evidence indicating any gas hydrate dissociation process. Other factors include soil disturbance

at deeper depths where stress relief can play an important role. The samples were retrieved using rotary

drilling advanced piston core (APC) and extended core barrel (XCB). With these sampling methods, it

is reported that soil disturbance is negligible at shallow depths (Paull et al., 2000) but some cracks were

observed from X-ray tests (Tan, 2004).

The normal compression lines of these two clayey soils are plotted on typical compression curves of

reconstituted clays with different plasticity indices in Figure 23. Both soils exhibit large compressibility

similar to reconstituted clay with plastic index of around 80%. This large compressibility may be due to

natural soil structure that the soils possess. The values of sensitivity (undisturbed/remoulded undrained

shear strength ratio) estimated from vane shear and pocket penetrometer tests were around 4.0–6.8. Soils

with such sensitivity values can increase the compressibility significantly (e.g. Mitchell & Soga, 2005).

Swelling index, C_s was around 0.15 and 0.06 for Blake Ridge and Hydrate Ridge samples, respectively.

In general, both samples did not show an appreciable swelling potential because montmorillonite is not the

main clay mineral. The recompression index, C_r is 0.037–0.064 for Hydrate Ridge samples (Tan, 2004).

6.3 Shearing

6.3.1 Sandy methane hydrate soils

6.3.1.1 Drained behaviour - Natural sediments

Isotropically consolidated drained triaxial compression tests were performed on natural sandy methane

hydrate samples retrieved from Nankai Trough by Masui et al. (2006). The hydrate saturations var

ied between 7.7 and 37.6%. The specimens of 50 mm diameter and 100 mm height were consolidated

isotropically at cell pressure of 10 MPa and back pressure of 9 MPa. Hence, the initial consolidation
Axial strain (%) -2 0 2 4 6 8 Deviator stress (kPa) -2 0 2 4 6 8 Volumetric strain (%) 65 413 54 513

Figure 24. Relationship between stress/volumetric strain-axial strain for 65, 413, 54 and 513 at 37.6%, 22.5%,

9.38% and 7.70% methane hydrate saturation respectively (Masui et al., 2006).

pressure was 1 MPa. The consolidated specimens were then sheared in compression at an axial strain

rate of 0.1%/min. The tests were performed in drained conditions and at temperature of 5 °C. Figure 24 shows the measured deviator stress-axial strain and volumetric strain-axial strain rela

tionships of the samples with different methane hydrate saturations. The soil exhibited strain softening

behaviour for Sample 65, which had the largest hydrate saturation of 37.6%. The peak stress was 6.11 MPa

at axial strain of 4.0%. The soil initially showed contractive behaviour, but then dilated at axial strain of

approximately 2%. The maximum dilation occurred at around peak stress level. The samples with lower

hydrate saturations exhibited smaller peak stress and less dilation at a given axial strain. The drained compression strength of the Nankai Trough specimens varied from 3.52 MPa to 6.11 MPa

when they were isotropically consolidated at effective stress of 1 MPa prior to shearing. The strength

increased with hydrate saturation from 3.52 MPa ($S_h = 7.70\%$) to 6.11 MPa ($S_h = 37.6\%$). The secant

Young's modulus at 50% failure stress (E_{50}) ranged between 139 MPa to 369 MPa and E_{50} generally

increased with hydrate saturation as shown in Figure 25. The Poisson's ratio at 50% failure stress (ν_{50})

was evaluated to be 0.10–0.19. No apparent relationship was found between ν_{50} and hydrate saturation. Anisotropically consolidated drained triaxial compression tests were also performed on natural Nankai

Trough sandy methane hydrate samples. To examine the hydrate cementation effects, tests were also

conducted on natural soil samples, in which the hydrate was dissociated by reducing the pore pressure.

Prior to drained shearing, the test specimens were consolidated by applying axial and radial total stresses

of 12 MPa and 10 MPa, respectively. The back pressure was 9 MPa. Hence the initial effective axial stress

and radial stresses were 3 MPa and 1 MPa, respectively; it was assumed that the lateral earth pressure

coefficient K_0 in the field is 0.5. All specimens had initial methane hydrate saturation between 24.8%

and 35.5%. The specimens were then sheared in drained conditions. Figure 26 shows the measured deviator stress-axial strain-volumetric strain behaviour. The drained

compression strength of the natural hydrate specimen (T2) was approximately 20 MPa. For the dissociated

specimens (T1D, T3D, T4D), the drained compression strengths reduced to 3–5 MPa, which are 1/4–1/3

of the strengths measured for the hydrate specimen. For the natural hydrate specimen, the volumetric

behaviour during shearing was small contraction at the

beginning but then dilation at larger strains. For

the dissociated specimens, contraction was observed throughout the shearing. Cemented soils also exhibit higher rate of dilation compared to non-cemented soils at low confining

pressure due to highly interlocking of the large soil particles/aggregates (Lade & Overton, 1989). With

increasing confining pressure, the dilation characteristic diminishes. Coop & Airey (2003) illustrate that

the effect of cementation is controlled by the location of the yield envelope of cemented soil relative to

the stresses required to cause yielding of equivalent uncemented soil; at low stress, it will increase the θ 0.1 0.2 0.3 0.4 Methane hydrate saturation 100 150 200 250 300 350 400 Young modulus, E 50 (MPa) 513 54 413 65

Figure 25. Relationship between E 50 with methane hydrate saturation (Masui et al., 2006). θ 2 4 6 8 10 12 14 16 18 Axial strain (%)

0

4

8

12

16

20

24

D

e v

i a

t o

r s

t r e

s s

(M

P a

) T4D (24.8%) T1D (35.5%) T3D (31.3%) T2 (S MH =31.8%)
Axial strain (%) -3 -2 0 2 4 6 8 10 12 14 16 18 -1 0 1 2 3
4 V o l u m e t r i c s t r a i n (%) T4D (24.8%) T1D
(35.5%) T3D (31.3%) T2 (S MH =31.8%) (a) (b)

Figure 26. Difference of (a) drained compressive strength
and (b) volumetric response between hydrate sediment

and non-hydrate sediment; hydrate sample (T2) and
dissociated sample (T1D, T3D, T4D) (Data from Mitsubitshi
Heavy Industry, MHI).

initial stiffness and shear strength; at intermediate
stress, it will only increase the initial stiffness; and at

high stress, the behaviour returns to non-cemented soil.

These results suggest that the presence of methane hydrate
has profound effects on its shear resistance

and dilation behaviour; that is, it increases the shear
resistance and enhances dilation, rather similar to

cemented sands.

6.3.1.2 Drained behaviour - Synthetic sediments

(1) Effect of hydrate saturations

For a given confining pressure, drained compression
strength increases with methane hydrate saturation.

However, the growth pattern of methane hydrate in soil
pores influences the rate of peak stress increase.

Masui et al. (2005a) prepared synthetic methane hydrate
specimens that have different hydrate formation 0 2 4 6 8
10 12 14 16 Axial strain (%) (b)(a) 0 1 2 3 4 5 6 7 8 9 D e
v i a t o r s t r e s s (M P a) 40.9% S MH = 67.8% 50.1%
26.4% 0 2 4 6 8 10 12 14 16 Axial strain (%) 0 1 2 3 4 5 6
7 8 9

D
 e v
 i a
 t o
 r s
 t r e
 s s
 (M
 P a
) 0.0% 25.7% 40.1% S M H = 55.1%

Figure 27. Effect of methane hydrate saturation on compressive strength (a) strong bonds (b) weak bonds (Masui

et al., 2005a). Methane hydrate saturation (%) 0 0 20 40 60
 80 4 8 12 16 20 M a x i m u m d e v i a t o r s t r e s s (M P a) Strong bonding (Ebinuma et al., 2005) Strong
 bonding (Masui et al., 2005a) Weak bonding (Ebinuma et al.,
 2005) Weak bonding (Masui et al., 2005a)

Figure 28. Peak strength versus methane hydrate saturation (Ebinuma data at confining pressure of 3 MPa, whereas Masui data at confining pressure of 1 MPa).

patterns in the soil pores. Samples with strong contact
 bond (Model C) and with weak contact bond

(Model A) were created. The methods of preparation are
 discussed in the previous section. Toyoura sand

was used as host sand. The specimens were initially
 consolidated isotropically to a cell pressure of 9.0 MPa
 and back pressure

of 8.0 MPa. Hence the initial effective stress was 1 MPa.
 They were then sheared at constant rate of

0.1%/min in drained conditions. Figure 27 shows the drained
 triaxial compression test results for;

(a) Model C with strong bonds and (b) Model A with weak bonds. Both models show increase in

strength and stiffness with hydrate saturation. The peak strength (i.e. maximum deviator stress) increases with hydrate saturation. However, the rate

of increase was different depending on growth pattern as shown in Figure 28 (Ebinuma et al., 2005).

For hydrate soils with strong bonds, the strength increased steadily with hydrate saturation. Hydrates Methane hydrate saturation (%) 0 0 10 20 30 40 50 60 70 80 200 400 600 800 1000 1200 Young modulus, E 50 (MPa) Strong bonding Weak bonding

Figure 29. Relationship between Young's modulus at 50% stress level (E 50) and methane hydrate saturation at

confining pressure of 1 MPa (Masui et al., 2005a).

formed at particle contacts, contributing to the increase in strength even by small amounts of hydrate.

The initial stiffness also seems to increase with hydrate saturation.

For hydrate soils with weak bonds, the strength increased only when the hydrate saturation was

more than about 25%, as shown by the data of Masui et al. (2005a) in Figure 28. Since hydrates were

preferentially formed inside the soil pores rather than at particle contacts, they occluded the pores but

did not contribute to the strength of the soils when the amount was small. It is hypothesized that the

shear resistance is primarily due to grain-grain interactions and the hydrates were more or less floating

inside the soil pores. As hydrate saturation increased more than 25%, the strength suddenly increased.

This is possibly because the stiffness/strength of hydrates themselves started to contribute to the strength

of the hydrate-soil mixture since they are now in contact

with the soil particles. The specimens had

similar initial stiffness values, suggesting that the initial stiffness is governed by the stiffness of the

host sand material. The contribution of hydrates to stress-strain response appeared only at larger strains.

Examining the data of Ebinuma et al. (2005) in Figure 28, both model type specimens exhibited strength

of more than 15 MPa when hydrate saturation was approximately 40%. This is about two times of the

peak strength of soils without hydrate. The greater strengths of Ebinuma et al.'s data compared to Masui

et al.'s data are due to different confining pressures applied (see next section).

Young's modulus at 50% stress level (E_{50}) increased with methane hydrate saturation as shown in

Figure 29 (Masui et al., 2005a). The data is for effective confining pressure of 1 MPa (total confining

pressure of 9 MPa and back pressure of 8 MPa). The trend of increase in E_{50} with hydrate saturation is

consistent with that observed in the drained compression strength as discussed before.

(2) Effect of confining pressure

For a given methane hydrate saturation, confining pressure also influences the mechanical behaviour.

Figure 30 shows the deviator stress-axial strain responses of synthetic methane hydrate specimens for

hydrate saturation of 34.0% (Masui et al., 2005a) and hydrate saturation of 38.5% (Ebinuma et al., 2003)

consolidated isotropically at three different confining pressures. As confining pressure decreases, the

hydrate soil exhibits greater strain softening, which is of typical behaviour of medium dense sands. In

Masui et al. (2005) data, the drained compression strength

increased 42% when the confining pres

sure increased from 1 MPa to 2 MPa, whereas it increased 21% when the confining pressure increased

from 2 MPa to 3 MPa. Ebinuma et al. (2003) data's has an exceptionally high peak stress when com

pared to the data by Masui et al. (2005). Further discussion on this is made in the failure envelopes

section. 0 2 4 6 8 10 12 14 16 Axial strain (%) 0 2 4 6 8 10 12 14 16 18 20 22 D e v i a t o r s t r e s s (M P a) 3 MPa 2 MPa 1 MPa 0.5 MPa 3.0 MPa Effective confining stress = 3.5 MPa

Figure 30. Toyoura cemented hydrate sand with ~34% S MH (solid line: Masui et al., 2005a) and ~38.5% S MH

(dashed line: Ebinuma et al., 2003). Methane hydrate saturation 0 0 0.1 0.2 0.3 0.4 0.5 0.6 2 4 6 8 10 12 D e v i a t o r s t r e s s (M P a) Effective confining stress 0.5 MPa 1.0 MPa 1.5 MPa 2.0 MPa

Figure 31. Effect of confining pressure and methane hydrate saturation on drained compression strength (Masui

et al., 2005b). Figure 31 shows an increase in drained compressive strength with hydrate saturation at four different

effective confining pressures (0.5, 1, 2 and 3 MPa). The data are produced using synthetic hydrateToyouura

sand specimens with strong bonds. The strength approximately doubles when the hydrate saturation

changes from zero to 50%. The rate of increase is greater when the confining pressure is smaller. The variation of Young's modulus at 50% failure stress (E_{50}) with effective confining pressure is

shown in Figure 32 for different hydrate saturations (Masui et al., 2005b). For soils with no hydrate,

E_{50} increases more or less linearly with effective confining pressure, as often observed in uncemented

sands. For hydrate soils with saturations of 25% or above, an increase in E_{50} is observed when the Effective

confining pressure (MPa) 0 0.1 2 3 4 200 400 600 800 Young's modulus, E_{50} (MN/m²) $S_{MH} = 0.0\%$ $S_{MH} = 24.3-26.5\%$ $S_{MH} = 33.7-34.8\%$

Figure 32. Variation of Young's modulus at 50% failure stress (E_{50}) with effective confining pressure (Masui et al.,

2005b).

confining pressure was increased from 0.5 MPa to 1 MPa. Above 1 MPa, however, E_{50} is approximately

independent of confining pressure; that is, the stiffness of the soil is governed by the strong bonds of

hydrate rather than by soil particle interaction.

6.3.1.3 Undrained behaviour - Natural sediments

Sandy soils usually deform in drained conditions because of large permeability of the soils. However,

undrained conditions can occur for sandy methane hydrate soils in certain scenarios. These include very

rapid drawdown of borehole pressure, limited drainage due to surrounding low permeability clay layers,

and a sudden landslide event or earthquake.

Winters et al. (1999a) performed undrained triaxial compression tests on natural gas hydrate bearing

samples. They were recovered from Mallik 2L-38 between depths of 898 and 913 m in sand hydrate

bearing sediment of Mackenzie-Bay sequence with 640 m of permafrost above it. Undrained triaxial

compression tests were performed on a hydrate soil specimen (GH062) and soil specimens in which

the hydrates were dissociated (GH058, GH059 and GH60). The specimens were about 70 mm diameter

and 130 mm length. One sample (GH058) was subjected to a cell pressure of 12 MPa and back pressure

of 9 MPa prior to undrained shear, but unfortunately

perforation of membrane occurred. As a result, a lower effective confining pressure of 0.25 MPa was applied to the rest of the samples. Considering the original depth of the samples, this initial effective stress state is not representative as its in-situ stress condition; the specimens are probably in heavily overconsolidated state prior to shearing.

The test results are presented in Figure 33 (Winters et al., 2002). The presence of gas hydrate in the sediment increased the undrained shear strength dramatically when compared to the undrained shear behaviour of specimens without hydrate; the undrained shear resistance of hydrate soil specimen at 5% axial strain is more than five times larger than that of non-hydrate soil specimen. The development of excess pore pressures during undrained shearing is shown in Figure 33(b). The hydrate soil specimen exhibits large negative excess pore pressures, whereas the non-hydrate soil specimens show much smaller negative excess pore pressures. The sample with hydrate exhibited dilation tendency while the samples without hydrate showed rather limited dilation tendency. The greater dilative tendency of the hydrate soil specimen is consistent with the drained behaviour of hydrate soils discussed in the previous section.

The large undrained shear strength measured for the hydrate soil specimen from the Mallik site is due to the following two reasons: (i) increase in bond strength or shear resistance by hydrates themselves, and (ii) increase in effective confining stress during undrained shearing due to dilative tendency of the methane hydrate soil. The dramatic increase in undrained shear strength observed in this particular test

series is partly due to small initial confining pressure applied, which leads to large dilative tendency

during undrained shearing. As discussed in the previous section, the dilation is suppressed when larger σ_3 is applied. Figure 33 shows the shear stress (a) and pore water pressure change (b) versus axial strain for GH062, GH060, GH059, and GH058 samples. The samples were sheared after the gas hydrate dissociated and GH062 was sheared

with

hydrate

with

no ice

present

from

Mallik

2L-38; (a) shear strength (b) pore pressure changes with axial strain

(after Winters et al., 2002). Figure 34 shows the shear stress (a) and pore water pressure change (b) versus axial strain for GH062, GH060, GH059, and GH058 samples. The samples were sheared after the gas hydrate dissociated and GH062 was sheared

with hydrate with no ice present from Mallik 2L-38; (a) shear strength (b) pore pressure changes with axial strain

(after Winters et al., 2002). Figure 34 shows the shear stress (a) and pore water pressure change (b) versus axial strain for GH062, GH060, GH059, and GH058 samples. The samples were sheared after the gas hydrate dissociated and GH062 was sheared

with hydrate with no ice present from Mallik 2L-38; (a) shear strength (b) pore pressure changes with axial strain

(after Winters et al., 2002). Figure 34 shows the shear stress (a) and pore water pressure change (b) versus axial strain for GH062, GH060, GH059, and GH058 samples. The samples were sheared after the gas hydrate dissociated and GH062 was sheared

with hydrate with no ice present from Mallik 2L-38; (a) shear strength (b) pore pressure changes with axial strain

(after Winters et al., 2002). Figure 34 shows the shear stress (a) and pore water pressure change (b) versus axial strain for GH062, GH060, GH059, and GH058 samples. The samples were sheared after the gas hydrate dissociated and GH062 was sheared

with hydrate with no ice present from Mallik 2L-38; (a) shear strength (b) pore pressure changes with axial strain

6.3.1.4 Undrained behaviour - Synthetic sediments

The chemical composition of hydrate also influences the mechanical behaviour of hydrate soils as the

stiffness/strength of contact bonds is governed by the hydrate themselves. For example, Yun et al. (2006)

created hydrate soil samples by forming hydrates of tetrahydrofuran (THF) in fine sand ($d_{50} = 0.1-$

0.12 mm), which has similar particle size as Toyoura sand. Dry soil and predetermined amount of THF

solution was mixed - closed to the Model A preparation method. Isotropic consolidation test was carried

out until predetermined effective confining pressure was attained. Specimens were then frozen to -10°C

to create hydrates (specimens of 0% hydrate-filled porosity were not subjected to freezing). The hydrate

specimens were stabilised for another 12 hours. The specimens with different hydrate saturations were

consolidated at three different confining pressures (0.03, 0.5 and 1 MPa) prior to undrained shearing. Figure 34 shows the results of the undrained triaxial compression tests. At low THF hydrate saturation,

the increase in strength was limited. When the THF hydrate saturation exceeded 45%, the drained

E /

$\sigma_c' = 0.03 \text{ MPa}$ 0.01 0.1 1 10 Axial strain (%) 0.01 0.1 1 10 100 0% 50% S THFH = 100%
 $\sigma_c' = 0.5 \text{ MPa}$ 0.01 0.1 1 10 Axial strain (%) 0.01 0.1 1 10 100 0% 50% S THFH = 100%
 $\sigma_c' = 1.0 \text{ MPa}$ 0.01 0.1 1 10 Axial strain (%) 0.01 0.1 1 10 100 0% 50% S THFH = 100%

Figure 35. Stiffness degradation curve for fine sand for different confining pressures (redrawn from Yun, 2005).

Percentages indicate THF hydrate saturation.

compressive strength started to increase with hydrate saturation. Berge et al. (1999), who used Freon

11 (CCL 3 F) as the hydrate former, report the hydrate saturation value of 35% as the start of strength

increase. Yun et al. (2006) made hydrate soil samples with 100% hydrate saturation (i.e. pore fully filled

with hydrate). The data shown in Figure 33 indicates that there are two regions of stiffness: initial high

stiffness up to axial strain of 1% followed by lower stiffness before peak strength.

The effect of THF hydrate saturation on stiffness was also greater at lower confining pressures.

Figure 35 shows the stiffness degradation curves. For the hydrate samples, the stiffness consistently

dropped suddenly at axial strain of 1%. Hence, the initial yielding is assumed to be related to breakage

of THF hydrate.

6.3.1.5 Failure envelopes

Figure 36 indicates the stress states at failure for synthetic sandy hydrate bearing sediment with strong

bonds (Fig. 36(a)) as well as natural sandy hydrate bearing sediment from Nankai Trough (Fig. 36(b)).

Soils with similar hydrate saturations are grouped together and friction angle and cohesion intercept

were determined. The derived values were plotted against hydrate saturation as shown in Figure 37.

Large number of tests on synthetic samples at different confining pressures allowed determination of

friction angle and cohesion independently. Results indicate that the friction angle is approximately 31 °

and is independent of hydrate saturation. The cohesion increased with hydrate saturation. Limited data

were available for natural hydrate samples. Hence, it was assumed that the friction angle is independent

of hydrate saturation. The friction angle of the natural samples was evaluated to be 35° . The cohesion

increased with hydrate saturation and the trend was similar to that of the synthetic samples, which may

make sense if hydrate is the only contributor to the increase in cohesion (i.e. bonding between particle

contacts).

Natural samples data that contained 7.7–37.6% gas hydrate saturation are consistent with the trend

of synthetic samples of 0–55% gas hydrate saturation (Masui et al., 2005a; Klar et al., in press). There

0 2 4 6 8 10 12 14 16 s' (MPa) 0 2 4 6 8 10 12 t (MPa) $S_{MH} = 37-40\%$
 $S_{MH} = 0\%$ $S_{MH} = 20-30\%$ $S_{MH} = 30-40\%$ $S_{MH} = 50-60\%$
 $S_{MH} = 44\%$ $S_{MH} = 0-10\%$ Ebinuma et al. (2003) Masui et al. (2005a) Klar et al. (2006) (a) s' (MPa) 0 2 4 6 8 10 12 14 16 2 4 6 8 10 12 t (MPa) Masui et al. (2006) MHI data MHI data (dissociated) 37.6% 7.7% 9.4% 22.5% 31.8%
Line $S_{MH} = 37-40\%$ Line $S_{MH} = 30-40\%$ Line $S_{MH} = 20-30\%$
Line $S_{MH} = 0\%$ Line $S_{MH} = 50-60\%$ Natural samples (b)

Figure 36. Stress path for sandy hydrate sediment for (a) synthetic strong-bonded samples (b) natural samples from

Nankai Trough accompanied with line of saturation obtained from synthetic samples with methane hydrate saturation

in percentage. 0 0.1 0.2 0.3 0.4 Methane hydrate saturation

0 5 10 15 20 25 30 35 40 45 Friction angle, ϕ ($^\circ$) 0 0.5 1 1.5 2 Cohesion (MPa) Natural gas hydrate, friction angle Synthetic gas hydrate, friction angle Natural gas hydrate, cohesion Synthetic gas hydrate, cohesion

Figure 37. Relationship between cohesion and friction angle with methane hydrate saturation in methane hydrate

cemented type (Natural data from Masui et al., 2006; synthetic data from Masui et al., 2005a).

is a large discrepancy between Ebinuma et al.'s data and Masui et al.'s data. This is not due to different

strain rates were used; strain rate of 6%/min for the former, whereas strain rate of 0.1%/min for the

latter. However, if the tests are done at subzero conditions or creation of ice due to endothermic process

during dissociation process, the existence of ice will resulted in greater rate effects (Hyodo et al., 2005).

In other words, the effect of strain rate is more significant in ice rather than hydrate because hydrate

has different structural arrangement compared to ice. The main reason for the higher strength is due to

different medium used which will be discussed later.

The stress state at failure of the natural hydrate sample (hydrate saturation = 31.8%) presented in

Figure 26 is also plotted in Figure 36 (described as MHI data). The strength of this sample was excep

tionally high compared to other natural hydrate samples with similar hydrate saturations and it can be

considered that this data is an outlier.

It is interesting to note that the difference in test conditions between this sample and the other samples

is the medium used for pore fluid during the drained tests. For the specimen with exceptionally high

strength, triaxial test was performed using air as the pore fluid. The other triaxial tests were performed

with water as the pore fluid. It is therefore hypothesized that the observed difference in strength is due

to weakening of natural hydrate bonds by water when the samples were saturated. In addition, three data

points of gas hydrate dissociated samples (open circle symbol) are also plotted in the same figure and

they lie above the zero hydrate saturation line of synthetic data. Further investigation is needed.

Winters et al. (1999a)'s data from Mallik 2L-38 site shown in Figure 33 are plotted in Figure 38 along

with the failure envelopes evaluated from Nankai Trough

Figure 39. Relationship between dilation angle with methane

hydrate saturation. 0 2 4 6 8

10

12

14

16 0 2 4 6 8 Axialstrain (%) 0 2 4 6 8 10 12 14 16 0 2 4 6
8 Axial strain (%)

-1.5

-1

-0.5

0

0.5

1

1.5

2

2.5

3 0 1 2 3 4 5 6 Axial strain (%) -1.5 1 -0.5 0 0.5 1 1.5 2
2.5 3 0 1 2 3 4 5 6 Axial strain (%) Experiment Model
prediction Experiment D e v i a t o r s t r e s s (σ_1 σ_3
) M P a

D

e v

i a

t o

r s

t r e

s s

(σ

1
 σ
3
) M
P a
V o
l u
m
e t
r i c
s
t r a
i n
(%
) S M H =0% S M H =10% S M H =44% S M H =53% S M H =0% S M H =10%
S M H =44% S M H =53% S M H =0% S M H =10% S M H =44% S M H =53%
S M H =0% S M H =10% S M H =44% S M H =53% V o l u m e t r i c
s t r a i n (%) Model prediction

Figure 40. Drained triaxial test on synthetic methane hydrate sediment using Toyoura sand (Klar et al., 2006).

the data are from tests with effective confining pressure of 1 MPa. It is possible that the relationship

changes with confining pressure and further investigation is needed.

Klar et al. (2006) used a modified version of Mohr-Coulomb model, where cohesion and dilation angle

were a function of hydrate saturation but friction angle was kept as a constant value. Figure 40 shows

the comparison of the model prediction with the actual data produced from synthetic methane hydrate

Toyoura sand specimens. A relatively good agreement can be found. In summary, hydrate is believed to

affect more on cohesion rather than friction angle. A similar argument was made by Kleinberg & Dai

(2005). The dilation angle increases with hydrate saturation.

6.3.2 Clayey methane hydrate soils

6.3.2.1 Drained behaviour

No drained shear data is available for the clayey methane hydrate soils of Blake Ridge and Hydrate

Ridge. Intact hydrate samples were not available for triaxial testing. Drained behaviour is important to

assess the long-term deformation behaviour. Further investigation is needed.

6.3.2.2 Undrained behaviour - Natural sediments

No undrained stress-strain data is available for Blake Ridge and Hydrate Ridge sites. Intact hydrate

samples were not available for triaxial testing.

Natural clay samples from Blake Ridge were obtained with APC (Advanced Piston Core) and

XCB (rotary Extended Core Barrel). The shipboard measurements of undrained shear strength using

miniature vane shear apparatus and pocket penetrometer were performed. The vane shear is 12.7 mm

high $\times$ 12.7 mm diameter vane at a rotation rate of about 90 ° /min. The vane shear and pocket penetrom

eter results were in good agreement with each other and results indicated an underconsolidated state as

discussed previously. The sensitivity of the clay was 4-7 and similar sensitivity values obtained from the

samples retrieved by two coring techniques suggest that the sampling did not induce considerable soil

disturbance at least for the upper sediments - Figure 41 (Winters, 2000a).

In Hydrate Ridge Site 1244, undrained shear strength was determined using vane test. The normalised

average undrained shear strengths (s_u / σ'_{v0}) of the soils in shallow (<20 mbsf) and deep layers (>20 mbsf)

were 0.35 and 0.31, respectively, similar as Boston Blue Clay (Tan, 2004). It should be noted that the

undrained shear strengths of Blake Ridge and Hydrate Ridge soils may be over-estimated due to the fast

shearing rate adopted. The rate effect on undrained shear strength needs further investigation.

Depth (mbsf)	Undisturbed vane shear (kPa)	Disturbed vane shear (kPa)	Pocket penetrometer (kPa)
0	300	200	100
40	200	100	80
80	100	80	120
160	80	60	160

Figure 41. Undrained shear strength from vane shear and penetrometer at Site 995, Blake Ridge (Winters, 2000a). Conventional triaxial undrained compression tests were performed by Tan (2004) on Hydrate Ridge

soil without hydrates. The average friction angle was estimated to be 36 ° at shallow depths and 33 ° at

deep depths.

6.3.2.3 Undrained behaviour - synthetic sediments

Yun et al. (2006) created hydrate soil samples using different soil types (sand, crushed silt, precipitate silt

and kaoline). Tetrahydrofuron (THF) was used as hydrate former. The procedure of creating the samples is

more or less similar as the sand sample as discussed before. Specimens with different hydrate saturations

were created. Undrained triaxial compression tests were performed at different confining pressures

(0.03, 0.5 and 1 MPa). The mechanical behaviour of the sand hydrate specimens are already shown

in Figure 34. Figure 42 shows the measured stress-axial

strain relationships of the other hydrate soils,

whereas Figure 43 shows the stiffness degradation curves. All soils show increase in strength/stiffness

with hydrate saturation. Coarser soils exhibit greater effect of hydrate on stiffness. The hydrate kaolin

exhibited more ductile behaviour than the hydrate sand. Yun et al. (2006) suggested that high specific

surface area of the sediment may not exhibit yield point because it will have a thinner layer of cementation

attributed from hydrate and it is believed that yielding occurs due to the collapse of hydrate cementation.

The importance of specific surface to the mechanical and electrical properties of hydrate sediment has

been pointed out by Francisca et al. (2005). The differences in undrained shear strength for different soil types are presented in Figure 44. All

non-hydrate soils exhibit a linear increase in undrained shear strength with increasing effective confining

stress. However, this is not true for the hydrate soils. The undrained shear strength of hydrate sample

with high hydrate saturation (50% to 100%) is rather insensitive to effective confining stress.

6.4 Small-strain stiffness

Most of small strain stiffness values are obtained based on acoustic velocities from seismic methods

in the field and bender element and resonant column tests in the laboratory. Assuming the tested soils

to be an isotropic elastic material, the relationship of P-wave velocity V_p and S-wave velocity V_s are

given by $V_p = \sqrt{(K + 4G/3)/\rho}$ and $V_s = \sqrt{G/\rho}$ where G , K and ρ are shear modulus, bulk modulus

and density of the sediment, respectively. As stressed by Priest et al. (2005), the comparison between the

results from field and lab methods should be in the same range of frequency as it will affect the velocities

measured.

0 2 4 6 8 10 12 14 Axial strain (%)

0

2

4

6

8

10

12

14

16

18

20

22

24

(a) 0 2 4 6 8 10 12 14 Axial strain (%) 0 4 8 12 16 Deviator stress (MPa) (b) 0 2 4 6 8 10 12 14 Axial strain (%) 0 4 8 12 16 20 Deviator stress (MPa) (c) 0 2 4 6 8 10 12 14 Axial strain (%) 0 4 8 12 16 20 Deviator stress (MPa)

Figure 42. Effect of surface area (a) Kaolinite D 50 : 1.1, Surface area: 36 (b) Precipitated silt D 50 : 20, Surface

area: 6 (c) Crushed silt D 50 :20, Surface area: 0.113 using tetrahydrofuron (THF) as hydrate former (Structure II).

Strain rate of 0.1%/min for non hydrate sediment and 1%/min for 50% and 100% hydrate-filled sediment. Letters a,b

and c denote 0.03, 0.5 and 1 MPa confining pressures (after Yun et al, 2006).

Table 5 lists the typical stiffness and velocity properties for different types of sediment constituents.

Methane hydrate has slightly larger stiffness compared to ice. The actual stiffness and wave velocities

of methane hydrate soil depend not only on the amount of hydrate in the soil, but also on the growth

pattern of hydrate in the soil pores as illustrated in Figure 11.

6.4.1 Sandy methane hydrate soils

In Mallik 2L-38 (50-90% methane hydrate saturation), V_p and V_s measured by in situ well logging were

2500-3600 m/s and 1100-2000 m/s, respectively, in the sand-rich sediments as shown in Figure 45(a)

(Collett & Ladd, 2000). The exceptionally high velocities around 925 m depth correspond to hard

cemented dolomite-sandstone. The low V_p/V_s ratio (<1.5) at the depth around 1120 m (see Fig. 45(a))

indicate the presence of free gas. The trend of small-strain shear and bulk moduli with depth is shown

in Figure 45(b). Larger stiffness values are measured in the hydrate layer compared to the non-hydrate

layer. Poisson's ratio is about 0.39 in the permafrost and gas hydrate zones, whereas it is 0.44 in the

unfrozen soil (Walia et al., 1999).

Wave velocities generally increase with hydrate saturation as shown in Figure 46 for Mallik samples

(Dai et al., 2004). The figure also includes hydrate saturation dependent wave velocities that were

predicted by the hydrate formation models shown in Figure 11. In Model A, gas hydrate is suspended

in the pore fluid and the bulk modulus of pore fluid is

only affected. Therefore, V_p increases with gas

hydrate saturation, but V_s remains to be almost constant.
 In this case, V_s is independent of gas hydrate 0.01 0.1 1 10
 Axial strain (%) 0.01 0.1 1 10 100 1000 $E / \sigma_0' = 0\% 50\%$
 S THFH = 100% $\sigma_c' = 0.03\text{MPa}$ (A) Kaolinite 0.01 0.1 1 10
 Axial strain (%) 0.01 0.1 1 10 100 0% 50% S THFH =
 100% $\sigma_c' = 0.5\text{ MPa}$ 0.01 0.1 1 10 100 Axial strain (%)
 0.01 0.1 1 10 100 0% 50% S THFH = 100% $\sigma_c' = 1.0\text{ MPa}$
 0.01 0.1 1 10 100 Axial strain (%) 0.01 0.1 1 10 100 1000 E
 $/ \sigma_0' = 0\% 50\%$ S THFH = 100% $\sigma_c' = 0.03\text{MPa}$ (B)
 Precipitated silt 0.01 0.1 1 10 100 Axial strain (%) 0.01
 0.1 1 10 100 0% 50% S THFH = 100% $\sigma_c' = 0.5\text{ MPa}$ 0.01 0.1
 1 10 100 Axial strain (%) 0.01 0.1 1 10 100 0% 50% S THFH
 = 100% $\sigma_c' = 1.0\text{ MPa}$

Figure 43. Stiffness degradation curve for (a) kaolinite
 (b) precipitate silt (c) crushed silt percentages indicate
 thf

hydrate saturation (redrawn from Yun, 2005). 0.01 0.1 1 10
 Axial strain (%) 0.01 0.1 1 10 100 1000

$E /$

σ

$\sigma_0' = 0\%$ S THFH = 100% $\sigma_c' = 0.03\text{MPa}$ (C) Crushed silt 0.01
 0.1 1 10 100 Axial strain (%) 0.01 0.1 1 10 100 0% S THFH
 = 100% $\sigma_c' = 0.5\text{ MPa}$ 0.01 0.1 1 10 100 Axial strain (%)
 0.01 0.1 1 10 0% S THFH = 100% $\sigma_c' = 1.0\text{ MPa}$

Figure 43. Continued

saturation as shear modulus of pore fluid is zero. In Model
 B, sediment porosity reduces, whereas V_p

and V_s increase exponentially with gas hydrate saturation.
 Based on this, Dai et al. (2004) concluded

that this supporting matrix model matches the gas hydrate
 occurrence in Mallik site. However, it tends

to overestimate the wave velocities at higher hydrate
 saturations. In Model C, V_p and V_s increase from

lower hydrate saturations as they are in contact with grain
 sediment. This cementation theory is only

valid for porosities less than 40% and this cementation

growth of hydrate is unlikely in Blake Ridge as

the sediments in this region are highly unconsolidated (Ecker, 2001). In Model D, V_p and V_s increase

rapidly from low hydrate saturation but the increase is not as drastic as Model C (Dvorkin et al., 2000;

Ecker, 2001).

The wave velocities increase with confining pressure as well as methane hydrate saturation. As pointed

out by Winters et al. (2000), the effect of confining pressure on P-wave velocity is important in order to

differentiate the effects of gas hydrate formation and confining pressure. Figure 47 shows the change in

P-wave velocity with confining pressure for Mackenzie Delta samples. The $\log(V_p) - \log(\sigma'_c)$ slope of

the plot is around 0.05-0.07. The P-wave velocities are much greater than that of the water and hence

the pressure dependency is due to the relatively stiff soil skeleton, whose bulk modulus is larger than

water.

By performing resonant column tests on synthetic hydrate samples, Clayton et al. (2005) showed that

increasing hydrate saturation leads to a reduced dependence of shear stiffness on effective confining

pressure. This is inferred by a reduced curvature at low effective confining pressure of high hydrate

saturation samples as shown in Figure 48.

Clayton et al. (2005) presented the lower impact of hydrate bonding on bulk modulus rather than on

shear modulus as shown in Figure 49. The ratio between saturated bulk modulus and shear modulus

dropped from 15-30 to about 2 as hydrate saturation increased from 0% to 10%. This implies reduction

in V_p/V_s ratio as hydrate saturation increases.

In summary, P-wave velocity is sensitive to free gas and less sensitive to gas hydrate formation as

the bulk modulus of saturated hydrate bearing sand is dominated by the bulk modulus of pore fluid and

Effective confining stress (MPa)	0.01	0.1	1	10
Undrained shear strength (MPa) (A) Sand	0%	50%	100%	0.01
Effective confining stress (MPa) (A) Sand	0.01	0.1	1	10
Undrained shear strength (MPa) (B) Kaolinite	0%	50%	100%	0.01
Effective confining stress (MPa) (B) Kaolinite	0.01	0.1	1	10
Undrained shear strength (MPa) (C) Precipitated silt	0%	50%	100%	0.01
Effective confining stress (MPa) (C) Precipitated silt	0.01	0.1	1	10
Undrained shear strength (MPa) (D) Crushed silt	0%	100%		

Figure 44. Relationship between undrained shear strength with effective confining at different hydrate saturation

(%) for different soil (A) Sand (B) Kaolinite (C) Precipitated silt (D) Crushed silt (Yun et al, 2006).

Table 5. Properties of sediment constituents (modified from Mavkov et al. 1998). Density $K G V_p V_s$

Region	(g/cm ³)	(GPa)	(GPa)	(km/s)	(km/s)
Quartz	2.65	36.6	45.00	6.0	4.1
Clay	2.58	20.9	6.85	3.4	1.6
Pure methane hydrate	0.90	7.9	3.30	3.7	1.9
Water	1.03	2.4–2.6	0.00	1.6	0.0
Ice	0.92	6.7	2.60	3.3	1.7

soil skeleton. As a result, an increase in S-wave velocity provides a good indicator of the presence of

gas hydrate in relation to V_p if the hydrate soil is fully saturated. At Mallik site, however, a reduction in

V_p/V_s ratio was observed due to the presence of free gas.

6.4.2 Clayey methane hydrate soils

For clay-rich Blake Ridge sediment (<10% gas hydrate

saturation), V_p and V_s are approximately

1700 ± 200 m/s and 500 ± 100 m/s, respectively as shown in Figure 50(a). The variation of shear and

bulk moduli evaluated from the wave velocity data is shown in Figure 50(b). The measured velocities (a) Shear modulus ($\times 10^9$ Pa) 1150 0 2 4 6 8 10 12 1100 1050 1000 950 900 850
D e p t h (m b s f) 0 10 20 30 40 50 Bulk modulus ($\times 10^9$ Pa) (b)

Figure 45. Mallik 2L-38 (a) Well log data (after Collett, 2000), (b) Variation of shear and bulk moduli with depth

(Collett et al., 1999).

and the stiffnesses in the clay sediments are generally lower than those in the sand sediments presented in

the previous section. The P-wave velocity is close to that of water, suggesting relatively soft soil skeleton

of the clay.

Despite the free gas that is present at the interval 450 m in Blake Ridge, the P-wave velocity does

not show a noticeable reduction. Evidence from the in situ logs and seismic interpretation data made by

Guerin et al. (1999) indicate that gas hydrate coexists with free gas over the interval. Hence, their effects

on wave velocities compensated each other to result in apparent insensitivity to the hydrate layer. (ii) 0 0.2 0.4 0.6 0.8 1 Gas hydrate saturation 0.5 1 1.5 2 2.5 3 S w a v e v e l o c i t y , V_s (k m / s) A B D C 0 0.2 0.4 0.6 0.8 1 Gas hydrate saturation 1 2 3 4 5

P

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 , V
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 (k m
 / s) A B D C (i)

Figure 46. Model prediction for (i) P-wave velocity (ii) S-wave velocity versus gas hydrate saturation in Mallik

2L-38 with the notation of (A) pore filling (B) supporting matrix (C) cementation (D) coating (redrawn from

Dai et al., 2004). 0.1 1 10 Effective consolidation stress (MPa) 1000 10000 V p (m / s)

Figure 47. Relationship between P-wave velocity and effective confining pressure from Mallik-2L-38 (Data from

Winters et al., 2000). 0 400 800 1200 1600 2000 2400 100 1000 10000 S m a l l s t r a i n s h e a r m o d u l u s , G m a x (M P a) Initial degree of saturation = 36% 18% 10% 5% 4% 3% 2% 1% Dense Loose

Figure 48. Relationship between shear modulus G (logarithmic scale) and isotropic effective confining pressure

(after Clayton et al., 2005).

Figure 49. Relationship between bulk and shear modulus ratio as function of initial degree of saturation (after

Clayton et al., 2005).

6.5 Creep

No experiment has been conducted to characterise the creep behaviour of natural methane hydrate

bearing sediments. However, Parameswaran et al. (1989) and Cameron & Handa (1989) performed

uniaxial creep tests on synthetic THF hydrate-quartz sand

sediment, whereas Durham et al. (2003)

performed compression creep tests (constant applied stress) on pure methane hydrate and ice. The tests

by Durham et al. (2003) were conducted at confining stress between 50 and 100 MPa and gas pore

pressure of 10-15 MPa. Different deviator stresses were applied at different temperatures (260-287 K)

and (an approximately constant) creep rate was measured. Figure 51 shows that, for both methane hydrate

and ice, creep rate increases with deviator stress and temperature at confining pressure of 50 MPa. They

all concluded that the strength is rate sensitive and hydrate-sediment is stronger than ice-sediment. This

agrees well with the larger stiffness of hydrate compared to ice (see Table 5).

7 ENGINEERING PROBLEMS

7.1 Submarine landslides

Man made activities that alter the gas hydrate stability of natural hydrate soils are related to oil industries

where operations are extending into deeper water depth. The area near tension leg platform (TLP), where

tension pile and suction anchors are used for keeping the floating structure anchored to the seafloor, can

also cause a reduction in pressure as water is pumping out to create suction environment (Hovland &

Gudmenstad, 2001). When methane hydrate dissociates by pressure reduction, it returns to methane gas

and water. The occupancy volume of these two elements is larger than the original volume. As shown in

Figure 52, this volume expansion is greater at shallower depths because of relatively less counterweight

water pressure. If this situation happens within low permeability layers like clay sediment and there is no

fault or drainage path, it will create a layer of excess pore water pressure, which results in lower strength

than the surrounding. As a consequence, a slope instability can occur in offshore regions.

Slope instability caused by gas hydrate dissociation is seldom documented historically. The famous

slopes instabilities that may be linked with gas hydrate dissociation are in the Southwestern Africa, U.S.

Atlantic continental slope Norwegian continental margin, British Columbia fjords, Alaskan Beaufort Sea,

Caspian Sea and North Panama as indicated by Kvenvolden (1998). Unfortunately, concrete evidence to

be so is still not available because other factors like earthquake, tectonic uplift and geological features

(salt diapir) can be a reason as well. However, some studies indicate that the period of slope failure

coincides with the drop in the sea level during Pleistocene. Other evidences include the coincidence

of the BSR and slope failure location with enrichment of organic rich sediment deposits, presence of

pockmarks, craters, mud volcanoes and existence of free gas.

Figure 50. Site 995, Blake Ridge (a) Well log data (Chand et al., 2004), (b) Variation of shear and bulk moduli with depth (Guerin et al., 1999).

Figure 51. Comparison of pure methane hydrate and ice strength on Arrhenius diagram of log stress versus inverse

temperature. Lines indicate the strength of the sample at fixed strain rate (Durham et al., 2003).

Figure 52. Volume expansion amount variation with depth (after Dillon & Max, 2003).

7.2 Oil/gas well failures

Uncontrolled gas release will cause gas kicks, blowouts and

fires. At the same time, it reduces the friction

between oil well casing and the surrounding soil. The breakage of cementing occurs and an increase

in vertical drainage path leads to settlement of the surrounding soil (Lunne, 1994). If the excess pore

pressure does not dissipate, the casing buckles due to high pressure of the gas zone. Yakushev & Collett

(1992) and Collett & Dallimore (2002) report that there is potential hazards associated with gas hydrate

induced drilling in permafrost regions like Yamburg gas field in West Siberian Basin Russia, Prudhoe

Bay-Kuparuk River area in northern Alaska, Mackenzie Delta-Beaufort Sea region and Sverdrup Basin

of Canada. The released methane gas will percolate upwards due to its low density ($\sim 0.9 \text{ g/cm}^3$) relative to sea

water. The pockmarks features and 'soupy' sediment created by this gas released will make an unpleasant

situation for the pipeline layout as it will cause rupture/buckling of the pipeline if excessive free span

is exposed (Hovland & Gudmenstad, 2001). In addition, the pipeline itself will emanate heat to the

surrounding soil, worsening the situation of accelerated rate of methane hydrate dissociation. Corrosion

of steel is another problem induced; the combination of released gas methane and pore water containing

sulphate will form acidic fluids that are nuisance to metals. There are several measures that have been taken into account to minimise the drilling problems related

gas hydrate dissociation (Collett & Dallimore, 2002):

1. Drilling mud is cooled between 0 and 5°C in order to avoid hydrate dissociation.
2. Cement of high strength, good bonding, low curing time and low heat of hydration is used to support the casing.

3. High mud weight (0.86-0.90 g/cm³) or sometimes (1.1-1.15 g/cm³) is used to balance the gas hydrate control and lost circulation.

4. Chemical mud additives (lecithin surfactant) - exchange of fluid between gas hydrate and drilling mud preserving in situ gas hydrate.

7.3 Methane gas extraction for energy use

In order to extract methane from deep subsurface for commercial use, it must be dissociated from the

hydrate form. To do so, either pressure needs to be decreased or temperature to be increased. These

two approaches constitute the main methods for future planned extraction processes: (1) depressurised

wells - in which the well pressure is decreased to allow flow into the well and pressure reduction in

the surrounding soil for methane dissociation. (2) Thermal injection wells - in which hot water are

injected into the well in order to increase the temperature in the ground and allow for dissociation

and (3) inhibitor injection - injection of chemical fluid such as methanol to shift its thermodynamic

equilibrium. Schematic diagrams of these methods are shown in Figure 53. In economic perspective, depressurisation is the most promising method. Thermal injection method

is always criticised due to its substantial heat loss. Inhibitor injection method can reduce the temperature

of gas hydrate decomposition, increase temperature gradient and produce higher energy efficiency.

However, it is the most expensive method and thus only small quantity can be applied for cost-effective

usage. Natural or induced fractures are believed to enhance the gas production rate. Depressurisation with inhibitor injection method has been used in a permafrost hydrate site, Mes

soyakhka in northern Russia, which is known to be the first

gas hydrate site for production purpose. In

Figure 53. Schematic picture of gas hydrate production methods (a) depressurisation, (b) thermal injection, (c) inhibitor injection.

Mallik L-38 and 2L-38 reservoirs, preliminary analysis indicate depressurisation method to be the most

economically feasible production technique (Khairkhah et al., 1999). Further investigation has been

carried out in Mallik 5L-38 reservoir with depressurisation and thermal injection methods. Findings

revealed that combined methods are better than a single method. Other production methods include the

use of horizontal wells because they involve larger intersection area within gas hydrate zone (Dallimore &

Collett, 2005).

Problems that arise during gas production include reformation of gas hydrate or creation of ice

owing to its endothermic dissociation process, changes in permeability, strength and compressibility,

and gas channelling behind casing. More field trials and laboratory tests are needed to develop more

comprehensive understanding of different production methods in order to use extracted methane gas as

energy resources, which is currently the main driver of gas hydrate research.

ACKNOWLEDGEMENTS

The work is financially supported by MH21 Research Consortium in collaboration with the National

Institute of Advanced Industrial Science and Technology (AIST). The authors thank Drs T. Ebinuma,

K. Aoki, K. Suzuki, H. Minagawa and H. Narita of AIST for their advice and comments to the paper on

Nankai Trough data and Dr. K. Ookawa and Mr K. Takagi of Mitsubishi Heavy Industry for providing

the data shown in Figure 26.

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Natural Soils - Tan, Phoon, Hight & Leroueil (eds) © 2007
Taylor & Francis Group, London, ISBN 978-0-415-42691-6

Spatial variability of sensitive Champlain sea clay and an
application of stochastic slope stability analysis of a cut

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ABSTRACT: A stochastic slope stability analysis method is
proposed to investigate short-term stability

of unsupported excavation works in a soft clay deposit
having spatially variable properties. Spatial

variability of undrained shear strength is modelled by a
stochastic model that is the sum of a trend

component and a fluctuation component. The undrained shear
strength trend, which is also spatially

variable, is modelled by a random function. Slope stability
analyses are performed on the stochastic

soft clay model to investigate the contribution of spatial
variability of undrained shear strength to a

disagreement between high factors of safety computed from
deterministic methods for slopes that have

failed. Probabilities of failure as computed from the
stochastic analyses give a better assessment of failure

potential. Probability of failure values also correlate
with time delay before failure. This phenomenon

may be related to progressive failure or creep and to
pore-pressure dissipation with time.

1 INTRODUCTION

Short-term stability of unsupported excavation works is of great interest in geotechnical practice. An

accurate evaluation of this type of stability is intimately dependent upon the applied analysis methods

and the representation of soil shear strength given as an input. Undrained shear strength as measured by

field vane testing is commonly used as an input for soil shear strength in total stress analyses. However,

field undrained shear strengths, especially on sites involving soft, sensitive clay deposits, often present

strong spatial variability (Soulié et al. 1990, Chiasson et al. 1995). In addition, estimates of undrained

shear strengths at locations where no measurements are available pose many questions. These two factors

cannot be considered effectively in traditional deterministic total stress analyses, except, maybe, by the

experience of the designer.

Through detailed back analyses of a number of field embankments, Bjerrum (1972) found that direct

use of undrained shear strength in total stress analyses overestimated the level of stability of embank

ments. Cases of cut slope failures were reported where a prior stability analysis gave factors of safety

that were greater than 1.0 (Bjerrum 1972, Lafleur et al. 1988). Many explanations for the disagree

ment between site observation and stability analysis have been postulated such as "time effect" and

"anisotropy" (Bjerrum 1972) or "progressive failure" (Dascal & Tournier 1975). "Spatial variability"

(Soulié et al. 1990, Chiasson et al. 1995) and uncertainty involved in estimation of undrained shear

strength could be other factors that are responsible for this disagreement.

As an attempt to model uncertainty and variability of soil shear strength, reliability analyses consider

shear strength as a random variable. A probability of failure or reliability index is obtained as a supplement

to the deterministic factor of safety (Duncan 2000).

Spatial variability of soil shear strength is, however,

not considered in a reliability analysis, except by ad hoc approaches (El-Ramly et al. 2002). Using a

stochastic model for spatial variability of undrained shear strength (Chiasson et al. 1995), Chiasson &

* chssp@umoncton.ca 10 20 30 40 50 60 70 80 90 100 110 120
130 140 150 0 10 20 30 40 50 60 70 80 90 100 110 120 7 10
9 8 13 14 15 11 23 24 25 26 27 18 19 20 21 22 16 1 2 17 3 5
6 4 A B C 1 D F N C 2 E C 3 45 ° 18 ° 34 ° 27 ° Vane
profile A, B, ..., Slope failure scars 12

Figure 1. Site plan of St-Hilaire test excavation showing locations of vane profiles and slope failure scars (capital

letters A through F). Scale in metres.

Chiasson (2000) suggested a stochastic slope stability method to evaluate the failure risk rather than the

deterministic factor of safety. Undrained shear strength values were modeled as outcomes of a stochastic

process. Based on a well recorded test excavation work in spatially variable soft clay, this paper presents a

model for the spatial variability of undrained shear strength that combines a trend component with a

stochastic fluctuation (or residual) component. A random trend model for vane strength is also developed.

The stochastic slope stability method is applied to investigate: (1) how spatial variability and uncertainty

of shear strength of a soil may be modeled and (2) how spatial variability of shear strength can explain

the disagreement between high deterministic factors of safety computed from a total stress analysis and

the fact that slope failures were observed at the site.

2 SUBSOIL AND SITE DESCRIPTION

To investigate short-term stability of unsupported cuts in soft sensitive clay deposits of eastern Canada,

a test cut was excavated at a site near Saint-Hilaire, approximately 50 kilometres to the east of Montréal,

Canada (Lafleur et al. 1988). The crest of the excavation formed a 60 m by 60 m square in plan. It had

four slopes: 18 ° (south slope), 27 ° (north slope), 34 ° (east slope), and 45 ° (west slope). Excavation

depth was of 6 m on the 45 ° west slope side and 8 m for the three other slopes (Figure 1). The site is

underlain by three distinctive soil layers (Figure 2). From the top, a thin 0.6 m layer of fine uniform

beach sand overlies a 2.0 to 3.0 m layer of weathered and fissured clay. Below, lies 30 m of sensitive and

lightly overconsolidated Champlain Sea clay. A borehole 50 m to the north-east of the cut was halted

when a confined aquifer composed of a permeable glacial till (silty sand with some gravel) was reached.

Installation of an open piezometer permitted to observe the presence of a small downward gradient

through the clay and drained towards Richelieu River by the underlying confined aquifer. An extensive subsoil exploration including vane tests (27 vane profiles as shown in Figure 1) was

carried out to determine the undrained shear strength of the clay deposit. Vane profiles extend up to 15.5 m

in depth with readings spaced each 0.50 m. Horizontal spacing between profiles is typically 10.0 m. These field tests showed that undrained shear strength at this site has considerable statistical scatter

(Figure 2). In addition, the depth of the top layer of fissured and weathered clay was also found to vary

from 1.5 m up to 3.1 m. Such a layer is generally considered as a tension crack zone in stability analyses. During and after construction of the test excavation, eight (8) slope failures were observed. Major

slope failures occurred on the two steepest slopes (45 ° west slope and 34 ° east slope) while a small three

(3) m wide rotational slide was observed on the 27 ° south slope (Figure 1). 40 80 120 Vane shear strength (kPa) 1 2 3
IL 0.00 4.00 8.00

12.00

16.00 0 50 100 Limits and water contents (%)

D

e p

t h

(m

) wn wP wL

Figure 2. Typical geotechnical profile; wn, wP,wL and IL are respectively the natural water content, plasticity limit,

liquidity limit and liquidity index.

3 MODELING SPATIAL VARIABILITY

Stationary processes can be modelled with the help of classical geostatistics. These processes have values

that fluctuate around a constant mathematical expectation. When considering a trend, a more general

theory must be used. Matheron (1973) defined this theory as Intrinsic Random Functions of order k

(IRF-k). Delfiner (1976) gives a more practical description, and his paper is recommended for those

readers who wish more detailed explanations. Since the

object of this paper is not to discuss this theory

in depth, a general description will be made using classical geostatistics as a working basis.

3.1 Stochastic modelling theory

In classical geostatistics, in situ parameters are considered as Regionalized Variables (RV) where region

alized indicates that the variable is intrinsically related to its spatial location. Since this variable cannot

be known everywhere, and it cannot be easily represented by a mathematical function, it is convenient to

imagine the soil deposit as an outcome of a random process. This random process must take into account

the regionalized character of the variable. Therefore not any type of random process is acceptable. Sta

tionary Random Functions (SRF) takes into account the regionalized character of a soil deposit. Let us

then represent an in situ parameter $V(s)$ by the following:

where $M(s)$ is the mathematical expectation at a given location vector s in space (a constant in a stationary

process), $\varepsilon(s)$ is a zero mean stationary random function modelling fluctuations in function of location

vector s in space. When two outcomes $\varepsilon(s)$ and $\varepsilon(s+h)$ are close neighbours in space (small lag h), both

take similar values, indicating high interdependency. When values (viewed as outcomes of the stochastic

process) are far apart (large lag h), they tend to be independent of one another. Lag h where two values

start to be linearly independent is called the autocorrelation distance "a" or range of the spatial covariance

$C(h)$. This covariance is defined as:

In a natural process, such as a soil deposit, this spatial covariance is a priori unknown. It has to be

estimated. To do so, the preferred tool is the variogram $\gamma(h)$, since by definition, it filters constants,

and in stationary processes, the mean M of Equations 1 and 2 is a constant. Filtering the mathematical

expectation implies that fluctuation variances are estimated without bias. The variogram is defined as:

When the process is stationary, that is when $M(s)$ is a constant mean whatever s , then the variogram

simplifies to:

When a finite variance exists, that is the stochastic process can be modelled as a SRF of order 2, the

following relation links the variogram to the spatial covariance $C(h)$ at lag h :

where $C(0)$ is the covariance at zero lag and corresponds to the variance of the SRF of order 2 of the

random function that is modelling the natural process fluctuations. When a process is non stationary, the mean value $M(s)$ (Equations 1 and 2) is no more a constant.

This mean can be expressed, at least locally, by polynomials of degree k with unknown coefficients. To

characterize intrinsically $V(s)$, that is to estimate $C(h)$ of Equation (2), expressions of $V(s)$ need to be

manipulated, and these expressions need to be independent of the polynomial coefficients. In stationary processes, it was sufficient to use increments of order 0, $[V(s+h) - V(s)]$, since these

increments filter out the unknown constant mean. For polynomials of higher order k , increments of

order k need to be used to filter out the unknown polynomial trend. If this is an Intrinsic Random

Function of order k (IRF- k), the variance of these increments exists and it can be expressed by a

combination of generalized covariances (Matheron, 1973). In a stationary phenomenon IRF- $k = 0$, the

semi-variogram $\gamma(h)$ (half the variogram or semi-variogram) is a generalized covariance of order 0. A regionalized variable can be modelled as the sum of a stationary random function of order 2 with a

deterministic trend when the generalized covariance displays a sill (i.e. a horizontal threshold). In such

a case the relationship between the generalized covariance $K(h)$ and the covariance $C(h)$ is as followed:

where $K(\infty)$ represents the sill value of the generalized covariance. As a consequence, the relationship

between the generalized covariance and the variance $C(0)$ is:

If $K(0) = 0$, then the variance is simply the amplitude of the generalized covariance sill.

3.2 A spatial variability model for vane strength, the fluctuation component

Vane strength shows a clear tendency to increase with depth (Figure 2). Computations of the following

mean first-order increment permits to evaluate how the variable should be modeled, that is as a stationary

or non stationary random function. The latter case implies a model for the trend, be it linear or of a higher

order.

where $d(h)$ is the mean first-order increment for data spaced following a vertical axis by lag h and $v(z)$

is the measured variable of interest at depth z . Measurements $v(z)$ are in this case vane strengths c_u . By definition, the mean first-order increment filters constants. If random function $V(z)$ is defined as

$V(z) = \mu + \varepsilon(z)$, where μ is a constant mean and $\varepsilon(z)$ is a random fluctuation with zero mean and set

spatial covariance, then:

Thus, if the variable of interest fluctuates around a constant average value, that is it can be modeled as a

stationary random function, average increments will tend toward zero, regardless of lag h . On the other

hand, if the variable follows a trend, i.e. is it is non stationary, mean first-order increments will tend to

increase with lag h . Mean-first order increments for vane strength and piezocone measurements are not zero at St-Hilaire.

They increase linearly with lag h (Figure 3). Thus, vane strength values at St-Hilaire are not stationary.

Similar results are found for piezocone data (Chiasson et al. 1995). The strong linearity of the first-order

increments indicates that these measurements can be modeled as in Equation 1. Other more rigorous $d(h) = 1.571 h$ $R^2 = 0.9994$

0 2 4 6 8 10 12 14 16 -40 -20 0 20
 Lag h (m)
 Average
 increment
 in
 vane
 strength
 (kPa)
 versus
 lag h (m)

Figure 3. Mean first-order increments $d(h)$ for vane

strength measurements. 0 2 4 6 8 10 12 14 16 -40 -20 0 20
 40 ϵ_{cu} (kPa) Depth (m) Figure 4. Detrended

undrained strength in function of depth z .

methods exist to confirm the order k of the trend. Chiasson & Soulié (1997) and Cressie (1988) propose

to observe the behavior of $K_1(h)/h^{2l+2}$ when h increases. The lowest integer value l for which this ratio

tends towards zero, when h increases, corresponds to the order k of the trend.

Stationarity in space of the variance needs also to be evaluated. This can be evaluated by simply plotting

detrended undrained strength values (Figure 4). Other techniques to evaluate this include computing the

squared $k = 1$ increments and plotting in function of spatial location. If this squared increment appears

stationary whatever depth z , one can conclude that IRF- k theory is applicable. This test is not sufficient

to conclude though that the fluctuation variance is stationary. One then also needs to plot the general

covariance of order k , $K(h)$, in function of h (details given below). Stationarity of the fluctuation

component can only be concluded if $K(h)$ tends towards a sill value when h increases.

Presence of this trend excludes using classical stationary geostatistics and classical geostatistical tools

such as the spatial covariance or the variogram. Intrinsic random function of order $k = 1$ (IRF- $k = 1$)

theory is a suitable stochastic model for this case with its generalized covariance (Matheron 1973). The

generalized covariance is, as its name indicates, a generalization of the stationary spatial covariance.

Spatial structure of vane strength was thus analyzed with a nonparametric least square estimator of the

generalized covariance of order $k = 1$ (linear trended data). This estimator was developed by Chiasson &

Soulié (1997).

Vane strength measurements have an experimental generalized covariance which admits a sill

(Figure 5). As discussed in the preceding paragraph, the existence of such a sill indicates that vane

strength can be modelled by the sum of a stationary random function of order 2 (SRF-2) and a ver

tical linear trend. The random function part models vane strength fluctuations about the linear trend.

This random function part has a finite variance $C(0)$ corresponding to the sill $K(\infty)$ of the generalized

covariance $K(h)$. From Figure 5, the variance $C(0)$ for vane strength is of 24 (kPa)^2 . This gives a stan

dard deviation of 4.9 kPa. If this data would be normally distributed, 95% of fluctuation values would

be within $\pm 9.8 \text{ kPa}$. The range or autocorrelation distance "a" is of 2.0 metres and the experimental

generalized covariance passes through the origin with a linear behaviour near the origin (Figure 5). The

2.0 m autocorrelation distance indicates that data spaced by more than this distance are not correlated.

This means that a vane strength value has no influence on the fluctuation part of another vane strength

value if both values are separated by a lag of more than 2.0 m. In other words, fluctuations of such values

will have a conditional variance equivalent to the variance of the process. The linear behaviour near the

origin indicates that vane strengths evolve in space in a very erratic manner. The fact that the generalized

covariance passes through the origin indicates that the process has no nugget effect $C(0)$ (i.e. no white

noise). Vane strength profiles are thus continuous function (no jumps at $h=0$). No nugget effect also

implies that vane strength has no random experimental error. This does not exclude possible systematic

errors such as a faulty calibration of the testing apparatus. A spherical covariance (Deutsch & Journel

1998) was found to be a satisfactory model (Figure 5).
 -30
 -25 -20 -15 -10 -5 0 0 1 2 3 4 Lag h (m) $K(h)$ of vane
 strength (kPa) 2

Figure 5. Experimental generalised covariances $K(h)$

for vane strengths. 0 5 10 15 20 25 30 35 40 45 b_0 (kPa)
 0.2 0.6 1.0 1.4 1.8 2.2 2.6 3.0 b_1 (kPa / m) Figure 6.
 Scatter plot of intercept b_0 and slope b_1 of the
 undrained shear strength trend at vane profile locations. 0
 10 20 30 40 50 0 10 20 30 40 50 60 70 Lag h (m) Semi v a
 r i o g r a m (k P a) 2 Omnidirectional North-South
 East-West Model (36) (40) (28) (14) (6) (6)

Figure 7. Variogram analyses of linear trend intercept b_0
 and Gaussian model: sill $C(0) = 21$ (kPa) 2 ; effective
 range

$a = 55$ m (number of omnidirectional lag couples are in
 parenthesis).

3.3 Trend component

Preceding paragraphs have shown that vane strength
 increases with depth z . No analysis has yet been

made to evaluate if a trend also exists in the horizontal
 plane. Let the trend be expressed mathematically as:

where $B_0(x_i, y_i)$ is the intercept of the vertical
 trend, and $B_1(x_i, y_i)$ is the slope. Both coefficients
 are

constant along the vertical axis or depth z . Chiasson et
 al. (1995) estimated these coefficients by the

kriging trend method (Deutsch & Journel 1998) for a
 neighborhood that included all data within a vertical

profile. Thus, each profile yields its b_0 and b_1 set of
 coefficients according to Equation 11. Note that

capital letters indicate a random variable or random
 function while lower cases correspond to outcome

values. Both coefficients are only dependent of location (x_i, y_i) in plan. Scatter plotting of estimated

values b_0 and b_1 for all vane profiles at the studied site shows that b_0 and b_1 change considerably in plan

and are negatively correlated (Figure 6). A relationship exists between profile location in the horizontal

plane, and trend values. This scatter of estimated vane trends cannot be explained by only estimation

variance about a global site trend where $M(s) = B_0 + B_1 z$ (see Chiasson et al. 1995 for detailed analysis).

Thus, a model needs also to be developed in the horizontal plane. The model needs to consider the negative

correlation between B_0 and B_1 . One model studied by Wang & Chiasson (2006) views coefficients B_0

and B_1 as two correlated random functions. Structural analyses of the trend components show that the

spatial variability of B_0 can be modeled by a Gaussian model with a sill $C(0)$ of 21.0 (kPa)^2 , a range

of 55.0 m , and no nugget effect (Figure 7). Omnidirectional, North-South and East-West variograms

for b_1 display considerable scatter and display a high nugget effect (Figure 8). For modeling purposes,

the East-West variogram is preferred for the subsequent stochastic slope stability analyses, since most $0.05 \text{ } 0.1 \text{ } 0.15 \text{ } 0.2 \text{ } 0.25 \text{ } 0 \text{ } 10 \text{ } 20 \text{ } 30 \text{ } 40 \text{ } 50 \text{ } 60 \text{ } 70$ Lag $h \text{ (m)}$ S e m i v a r i o g r a m (k P a / m) ² Omnidirectional North-South East-West Model (36) (9) (14) (9) (40) (28)

Figure 8. Variogram analyses of slope b_1 of linear trend and Gaussian model: sill $C(0) = 0.12 \text{ (kPa/m)}^2$; nugget

effect $c_0 = 0.09 \text{ (kPa/m)}^2$; effective range $a = 55 \text{ m}$ (number of lag couples are in parenthesis).

lag couples are within the neighbourhood of the 34° and 45° slopes. The model used for b_1 is a Gaussian

model with a sill $C(0)$ of 0.12 (kPa/m)^2 , a range of 55.0

m, and a nugget effect of 0.09 (kPa/m)^2 . Random

functions B_0 and B_1 will be assumed bivariate normal. In particular the conditional distribution of B_1

given B_0 is normal with mean and variance:

In the above Equations 11 and 12, μ_{b0} , μ_{b1} , σ_{b0} , σ_{b1} , are the mean and standard deviation of random

variables B_0 and B_1 , respectively. $\mu_{b1|b0}$, $\sigma_{b1|b0}$, are the conditional mean and standard deviation of

random variable B_1 given B_0 . ρ is the correlation coefficient of random variables B_0 and B_1 .

4 STOCHASTIC SLOPE STABILITY METHOD

4.1 The upper bound method

The spatial variability or stochastic model developed in the preceding section permits to generate a set of

outcome maps of undrained shear strength for a selected cross-section. By performing stability analyses

on the set of maps, a set of factors of safety is obtained. These analyses are performed with an upper

bound method (Donald & Chen 1997, Wang & Chiasson 2006). Through construction of a kinematically

admissible velocity field, the upper bound method iteratively determines the factor of safety by solving a

work-energy balance equation. An optimisation method called random search simplex method searches

for the critical slip surface associated with the global minimum factor of safety (Donald & Chen 1997).

Theoretically, the factor of safety determined by the upper bound method is an upper bound value. It

should hence always be greater than or equal to the true factor of safety. However, Michaloski (1995)

and Donald & Chen (1997) have proved that the upper bound method is equivalent to limit equilibrium

methods using multi-slice, translational mechanisms of sliding masses, with the advantages of avoiding

complicated calculation of forces and a number of simplifying assumptions.

4.2 Definition of failure threshold F_f

Discrepancies between factors of safety calculated using undrained shear strengths and observed per

formances of large-scale cuts and embankments have long been recognised (Bjerrum 1972 and 1973,

Azzouz et al. 1983, Aas et al. 1986). Leroueil et al. (1990) revisited data from these sources and others.

They noted that the short-term stability of cuts in non-fissured clays, having liquidity indices greater than

0.4 and plasticity indices less than 50, could be estimated on the basis of undrained shear strength meas

ured with the field vane or from unconsolidated undrained triaxial (UU) tests when the pore-pressure $F_{fST} = 0.0107 IP + 0.7686 R^2 = 0.2387$
0.4 0.20 40 60 80 100
0.8 1.2 1.6 2.0 Plasticity index (IP) Computed factor of safety (F_{fST}) Vane Triaxial Bjerrum
Maximum 75th percentile Median 25th percentile Minimum Box plot of average Fos

Figure 9. Factors of safety obtained by total stress analyses of short-term failures (F_{fST}) versus the plasticity index.

Case where $IL > 0.4$ and $IP < 50$ (adapted from Leroueil et al. 1990).

redistribution is not significant. A least square line obtained from their data is plotted at Figure 9. Its

equation is: This line is close to the trend first proposed by Bjerrum (1972, 1973). At St-Hilaire, the average

plasticity index for a depth range of 2.5 to 8.0 m is $29.7 \pm 1.8\%$. Thus, based on the above equation,

slopes that have a computed, short term, total stress factor of safety of 1.087 will on average fail at

St-Hilaire (Figure 9). In this paper, this value is defined as the factor of safety at failure threshold (F_f).

If the computed factor of safety F_{os} is equal or less than F_f , the slope is in a state of failure.

4.3 Definition of probability of failure

For N cross-sectional outcome maps of undrained shear strength, there are accordingly N factors of

safety that are obtained. If there are M factors of safety less than the failure threshold F_f , a “Calculated

Probability of Failure” is defined as $P_{fc} = M/N$. If there are no factors of safety less than the failure

threshold F_f , or not enough for a reliable statistical prediction within the N outcomes, a statistical

distribution of the factor of safety needs to be estimated first. A “Nominal Probability of Failure” P_{fn}

is then defined as the probability of the event of $F_{os} \leq F_f$ based on the approximate distribution. For

example, if the factor of safety is assumed to be normally distributed, the nominal probability of failure

would be determined by:

In this paper, all values of probability of failure are of the calculated type.

4.4 Results from stochastic analyses

As shown in Figure 1, eight (8) slides occurred along 45 °, 34 °, and 27 ° slopes during and after excavation.

In order to explore the excavation behaviour in terms of cut depth, 2-D stochastic slope stability analyses

are performed for a number of cross-sections passing through vane profiles Su9, Su13, Su12, Su14 and

Su15 for the 45 ° slope, Su19, Su20, Su21 for the 34 ° slope, and Su25 for the 27 ° slope. These cross

sections are relatively well centred with observed failure scars. Water contents for the top fissured clay

and the underlying intact clay deposit are assumed to be 40% and 50% respectively. These are average

site values. Water content for the studied site ranges between 34% and 99%, with a standard deviation of

7%. Average water contents of 40% (top fissured clay) and 50% (intact clay) correspond to unit weights

Table 1. Results from stochastic slope stability analyses (random trend). Neighbouring Depth of Standard Probability of

Slope vane profile cut (m) Mean Fos deviation of Fos failure (%)

45 ° Su9 6 1.04 0.21 60.7 Su13 6 1.05 0.22 58.0 Su12 6 1.26 0.22 20.7 Su14 6 1.28 0.21 17.0 Su15 6 1.23 0.22 25.7

34 ° Su19 5 1.29 0.28 26.0 Su20 8 1.21 0.21 29.0 Su21 8 1.14 0.18 44.7

27 ° Su25 8 1.59 0.25 3.0 0.4 0.5 0.6 0.7 0.8 0.9 1 1.1 1.2 1.3 1.4 40 50 60 70 80 90 South towards north transect (m)
M e a n F o s 0% 20% 40% 60% 80% 100% P r o b a b i l i t y
o f f a i l u r e Mean Fos pf Su15 Su14 Su12 Su13 Su9 F f
= 1.09

Figure 10. Computed mean factor of safety and probability of failure along 45 ° slope axis.

of 18.2 kN/m³ and 17.2 kN/m³. Tension cracks are assumed full of water. Mean factors of safety, failure

probabilities, and other statistics for these selected cross-sections were computed for undrained shear

strength maps based on the random trend model (Table 1).

Excavation of the steepest 45 ° slope progressed from north towards south during a five-day period

(Lafleur et al. 1988). Slope failures also occurred chronologically in the same order once the excavation

reached a depth of 6 m. Two major slope failure zones can be identified; one is centred at cross-section

Su13, between cross-sections Su9 and Su12, the other is

between cross-sections Su12 and Su15. Mean

factors of safety at cross-sections Su9 and Su13 are 1.04 and 1.05 (Table 1). All these values are close

and below the failure threshold $F_f = 1.09$ (as defined earlier). However, mean factors of safety at cross

sections Su12, Su14, and Su15 are close to 1.26, which is greater than F_f (Table 1). Probabilities of

failure are 60.7% and 58.0% at cross-sections Su9 and Su13 and in the order of 17% to 25.7% at cross

sections Su12, Su14, and Su15 (Table 1). These analyses suggest that there are two distinctive slope

failure zones. This is in agreement with site observations (Figure 1). It is also interesting to note that

higher probabilities of failure are associated with the zone neighbouring cross-sections Su9 and Su13

where slope failures occurred immediately after the completion of excavation. Lower probabilities of

failure are associated to the zone between cross-sections Su12 and Su15 where slope failures occurred

3 days after completion of excavation. Variations of mean factor of safety and probability of failure for

cross-sections along the crest are shown in Figure 10.

Work on the 34 ° slope on the east side also progressed from north towards south. One larger failure

(scar A of Figure 1; 14 m wide by 23 m long) centred near vane profile Su19 occurred when the slope

was cut down to a depth of 5 m. The computed mean factor of safety and probability of failure at cross

section Su19 are 1.29 and 26.0% (Table 1). The other larger slide occurred at the designed excavation

depth of 8 m and is centred near cross-section Su21 (scar B, Figure 1). At this cross-section, a mean

factor of safety of 1.14 and a probability of failure of

44.7% are obtained (Table 1). Mean factors of 0.4 0.5 0.6 0.7 0.8 0.9 1 1.1 1.2 1.3 1.4 40 50 60 70 80 90 South towards north transect (m) M e a n F o s 0% 20% 40% 60% 80% 100% P r o b a b i l i t y o f f a i l u r e Mean Fos pf Su21 Su20 Su19 F f = 1.09

Figure 11. Computed mean factor of safety and probability of failure along 34 ° slope axis.

safety for the random trend model are higher and probabilities of failure are lower than expected for

cross-sections where slope failures have occurred (Figure 11). An average mean factor of safety closer

to the failure threshold $F_f = 1.09$ would have been expected and the average probability of failure for

both these slopes, which is 35%, should have been closer to 50%. At cross section Su20 where no slope

failure was reported, a probability of failure of 29% and a mean factor of safety of 1.21 are computed.

This is a rather high probability of failure for a slope that has actually never failed. Excavation of the 27 ° south slope advanced from east to west and necessitated 34 days, corresponding

to the total duration of the construction period. A small spoon shaped failure developed 26 days later (scar

F, Figure 1). The analysis gives a rather high value for the mean factor of safety (1.59). The probability

of failure is consequently low. The stochastic model gives a probability of failure of only 3.0% (Table 1).

5 DISCUSSION

5.1 Probability of failure and observed slope behaviour

Results generally yield high values of probability of failure where slope failures were located. An

exception to this rule is found at cross-section Su20, where no failure was observed even though the

probability of failure is rather high. This can be explained by two factors. First, debris from failures scars

A and B (Figure 1) created a berm at the foot of cross-section Su20, increasing the resisting moment.

Second, high undrained strength values were measured at depths of 10, 8 and 7 m at vane profiles Su19,

Su20 and Su21. These measurements indicate the presence of a zone of higher strength clay that extends

between these three profiles. The highest undrained strength values, reaching up to 70 kPa, are found

at profile Su20. Although it is unknown how this zone extends in the East-West direction, the fact that

the slope at cross-section Su20 was still standing after a number of years indicates that the zone of high

undrained shear strength clay is more than probably present within this section of the slope. This would

explain why the slope at Su20 is still standing even though the computed probability of failure is high.

5.2 Comparison of results from stochastic analyses and deterministic analyses

Deterministic slope stability analyses were performed at the same cross-sections to explore how these are

related to stochastic slope stability analyses. Undrained shear strength is determined by linear interpol

ation of vane profile measurements. For example, at cross-section Su13, the undrained shear strength map

corresponds to measurements obtained from vane profile Su13. Undrained shear strength on horizontal

transects at locations (or grid points) where no measurements are available are equal to vane profile

Su13 measurements at the same depths (no spatial variability in plan). These analyses are identified as

deterministic Fos based on measured undrained shear strength. In addition, kriging (Journel & Huijbregts

1978), a local estimation technique which provides the best

linear unbiased estimator from measured

Table 2. Comparison of factors of safety between stochastic and deterministic methods. Deterministic Fos Neighbouring Stochastic

Slope vane profile Depth (m) mean Fos Measured C u Kriged C u

45 ° Su9 6 1.04 0.94 0.93 Su13 6 1.05 1.16 1.15 Su12 6 1.26 1.34 1.31 Su14 6 1.28 1.32 1.36 Su15 6 1.23 1.33 1.36

34 ° Su19 5 1.29 1.29 1.39 Su20 8 1.21 1.34 1.46 Su21 8 1.14 1.33 1.34

27 ° Su25 8 1.59 1.65 1.53 0 20 40 60 80 0.8 1 1.2 1.4 1.6
Mean Fos P r o b a b i l i t y o f f a i l u r e (%)

Figure 12. Probability of failure in function of mean factor of safety.

data of the studied unknown characteristics, was also used to compute undrained shear strength at grid

points. These analyses are identified as deterministic Fos based on kriged undrained shear strength.

Stochastic mean factor of safety and deterministic factors of safety obtained from measured and kriged

maps of undrained shear strength are tabulated in Table 2.

The factor of safety obtained from the kriged undrained shear strength is comparable to that from the

measured undrained shear strength. However, stochastic mean factors of safety are generally smaller

than those from deterministic analyses. Note that in the zone of interest for slope stability analysis, the

kriged undrained shear strength is predominantly dependent of measured values from the closest vane

profile. This is why there is no significant difference between factors of safety based on measured or

kriged undrained shear strength.

5.3 Relation between probability of failure and mean factor

of safety

Mean factor of safety (expectation of the random variable Fos) and probability of failure are presented

to evaluate risk of instability of excavated slopes. The mean factor of safety from stochastic analysis is

designed to be the best linear estimation of the true factor of safety. Results show that it is generally

smaller but comparable to the deterministic factor of safety (Table 2). Mean factors of safety computed

for 45 ° , 34 ° and 27 ° slopes are generally higher than the failure threshold $F_f = 1.09$. Thus, it could be

concluded that, on average, most slopes should have been stable. The mean factor of safety, like the

deterministic counterpart, does not give a good assessment of potential slope failure occurrences at this

site. A factor of safety that is safe on average does not prevent that there may be instances where its value

will be lower than the failure threshold. The probability of failure, on the other hand, takes into account

the influence of spatial variability of undrained shear strength on the statistical distribution of the factor

of safety. The probability of failure is both a function of mean and statistical dispersion (measured by

standard deviation) of the factor of safety.

To further explore the relation between probability of failure and mean factor of safety, stochastic

slope stability analyses of 51 ° , 40 ° , 34 ° and 30 ° slopes following cross-sections Su9, Su13, Su12, Su14 0.2 0.4 0.6 0.8 1.0 1.2 1.4 1.6 1.8 Fos -4 -3 -2 -1 0 1 2 3 4 Expected Normal Value

Figure 13. Normal plot of factor of safety for cross-section Su13 (6 m deep cut).

and Su15 were performed. Plot of these analyses along with preceding results for the 45 ° and 34 ° slopes

(Figure 12) shows a non-linear relation between the mean factor of safety and probability of failure. This

good relationship between the probability of failure and the average factor of safety is a consequence of

the spatial variability model, which is the same throughout the site. The relationship between probability

of failure and average (or deterministic) factor of safety is thus site dependant. A different site will

display a different relationship. For the site investigated, Figure 12 can be used for a fast estimation of

probability of failure when the average factor of safety is estimated through a deterministic analysis.

5.4 Relation between probability of failure and delay for slope failure

Extensive stochastic slope stability analyses show that the factor of safety at this site follows the normal

distribution except for some deviation at the upper and lower tails of the statistical distribution (Figure 13).

This signifies that for a large number of failed slopes, the mean factor of safety from stochastic analyses

should be, on average, equal to the failure threshold F_f with a 50% of probability of failure. In the current

study, calculated mean factors of safety are on average close to or above the failure threshold $F_f = 1.09$.

The average probabilities of failure for the 45 ° and 34 ° slopes where failures were observed are of 36.4%

and 33.2%. Although there should be some statistical scatter in the mean probability of failure, it would

have been expected that the estimated values would approach 50%. There is, thus, some disagreement

between the computed probability of failure and failure observations. There are other possible sources

that may have contributed to this discrepancy, time between end of construction and failure being one of

them. For example, it is well known that the factor of safety of slopes excavated in clay is at its highest

value immediately after completion of construction (Bishop & Bjerrum 1960). As negative excess pore

pressures (generated by total stress release from soil excavation) dissipate, the factor of safety decreases.

Another contributing factor for the discrepancy with the average probability of failure could be attributed

to progressive failure due to strain softening with time. Plotting the probability of failure as a function

of slope failure delay in days shows that slopes with lower probability values failed following a longer

time delay after construction completion (Figure 14). Although data is scarce, two curves are plotted for

time delay to failure as a function of probability of failure. The 45 ° slope falls on Curve 1 and the 34 °

slope on Curve 2. Both of these two curves indicate that the lower the probability of failure, the longer

the delay for slope failure. However, the influence of progressive failure and pore pressure dissipation

or of another time dependant factor needs further investigation.

5.5 Spatial variability and statistical scatter of the computed Fos

Spatial variability is found to produce significant scatter in Fos (Figure 13) A histogram of Fos values

obtained from the random trend model for profiles from the 34 ° and 45 ° slopes, plotted over Bjerrum's

chart, shows clearly how the stochastic nature of the spatial variability influences statistical scatter of

the Fos (Figure 15). This figure also indicates that statistical scatter present in Bjerrum's chart may be

partly attributed to stochastic spatial variability of the sites used to develop the chart. 10 100 0 2 4 6 8 10 12
 Slope failure delay (day) P r o b a b i l i t y o f f a i l
 u r e (%) 45 degrees slope 34 degrees slope Curve 1 Curve
 2

Figure 14. Probability of failure as a function of delay for slope failure. 0.3% 0.7% 4.7% 11.3% 20.5% 22.7% 20.2%
 10.0% 6.3% 3.2% 0.2% 0.2 0 20 40 60 80 100 0.6 1.0 1.4 1.8
 2.2 Plasticity index (I P) C o m p u t e d f a c t o r o f
 s a f e t y (F f S T) Vane Triaxial Bjerrum This study

Figure 15. Statistical scatter of Fos values plotted over Bjerrum's chart for profiles from 34 ° and 45 ° slopes.
 Solid

line is short-term failure regression line as used in definition of failure threshold for this study.

6 CONCLUSIONS

Use of a geostatistics can provide a good model for the variability of a clay deposit when in presence

of a vertical trend. Vane vertical profiles, are continuous but erratic functions, and fluctuate around a

vertical trend. These fluctuations have a finite variance, and the vertical trend is variable over the site.

This later component is modelled as a random function. The proposed stochastic model for the spatial

variability of the studied site is the sum of a stationary random function of order 2 and of a random trend

function.

The stochastic variability model for vane strength is combined with a limit analysis method for

slope stability. This is proposed to investigate the short-term stability of unsupported excavation works

involving spatially variable subsoil. Stochastic slope stability analyses are performed to investigate the

contribution of the spatial variability of undrained shear strength to the disagreement between the high

factor of safety computed from deterministic methods and the fact that slope failures occurred. Analysis

results lead to the following conclusions:

- the spatial variability of the undrained shear strength has a significant influence on excavation stability;
- stochastic mean factors of safety are comparable to deterministic factors of safety;
- a correlation exists between probability of failure and delay in time for failure with lower probabilities

of failure indicating a longer delay before failure will occur. Progressive failure and pore pressure

dissipation may be responsible for this.

ACKNOWLEDGEMENTS

Funding for this research work was provided by grant OGP013720 of the National Science and Engin

earing Research Council of Canada (NSERC) and by the China National Natural Science Foundation,

grant No. 50539100 and No. 50509027. The authors are also grateful to the Université de Moncton

for its financial and technical support and to the China Institute of Water Resources and Hydropower

Research (IWHR).

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Characterisation and Engineering Properties of Natural
Soils - Tan, Phoon, Hight & Leroueil (eds) © 2007 Taylor &
Francis Group, London, ISBN 978-0-415-42691-6

Modeling the natural variability of over-consolidated clay
in

Adelaide, South Australia

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ABSTRACT: The properties of soil and rock are, as a result
of their formation, variable from one

location to another. In terms of geotechnical engineering
analysis and design, it is often desirable

to quantify this spatial variability. Due to the complexity
of this variability, models that have been

developed have necessarily focused on statistical
techniques. This paper presents a brief overview of

the methods commonly applied to modeling spatial, or
natural, variability of geotechnical properties.

Particular attention is given to random field theory and
geostatistics as these have been more widely

adopted and have shown the most promise. Following their
treatment, a case study is presented to

illustrate the application of these methods to geotechnical
engineering. Finally, the application of spatial

variability to various geotechnical engineering problems is
briefly discussed in order to demonstrate the

benefits of incorporating spatial variability in
geotechnical engineering analysis and design.

1 INTRODUCTION

Soils are inherently variable from one location to another.
This is known as spatial variability and is due

mainly to the complex and varied processes which influence

their formation and continual alteration.

These processes include: sedimentation; parent material; weathering and erosion; climate; topography;

organisms; structural defects - such as faults, folds, joints, fractures, slickensides and fissures; layering;

stress history; suction; and time. This paper is concerned with modeling the spatial variability of soils

and aims to present an overview of the available techniques in use through their application to an

over-consolidated soil in Adelaide, South Australia known as Keswick Clay.

Due to the often large and somewhat random nature of the spatial variability of soils, deterministic

methods generally fail to adequately quantify geotechnical properties. As a consequence, it is often

preferable, and more appropriate, to use methods based on statistical and probability theory. Since

the pioneering work of Lumb (1966), spatial variability modeling of soils has concentrated on three

techniques - random field theory, geostatistics and, more recently, fractals. Along with classical regres

sion analysis, these are each examined below, followed by their application to modeling the spatial

variability of the Keswick Clay.

2 SPATIAL VARIABILITY MODELS

2.1 Regression analysis

Classical statistics and regression analyses can be applied to many situations in geotechnical engineering

and the earth sciences which involve the analysis of bivariate and multivariate data. It is common, when

comparing two distributions, to display these in the form of a scatterplot. This is an x-y graph on which

the x-coordinate corresponds to the value of one variable, and the y-coordinate corresponds to the value

of the other variable. The scatterplot represents the correlation, that is the degree of dependence, between

the two variables. The correlation is quantified using the covariance and the correlation coefficient. The covariance, C_{XY} , between two sets of data $X = \{X_1, \dots, X_n\}$ and $Y = \{Y_1, \dots, Y_n\}$ is defined as:

where $\bar{X}$ = arithmetic mean of $X = n \sum_{i=1}^n X_i / n$; $\bar{Y}$ = mean of Y ; and n = number of data. The correlation coefficient, r , is defined as:

where σ_X = standard deviation of $X = \sqrt{1/n \sum_{i=1}^n (X_i - \bar{X})^2}$; and σ_Y = standard deviation of Y . When: $r=+1$ the data fit a straight line of positive slope, perfectly; $r=0$ the data exhibit no cor

relation; and $r=-1$ the data fit a straight line of negative slope, perfectly. As a consequence, as $|r|$

approaches 1, the data fit more closely to a straight line. Smith (1986) suggested that, for values of $|r|$

between 0 and 1, the following rough guide may be used:

- $|r| \geq 0.8$ strong correlation exists between X and Y , which can be assumed to be completely dependent;
- $0.2 < |r| < 0.8$ correlation exists between X and Y ;
- $|r| \leq 0.2$ weak correlation exists between X and Y , which can be assumed to be independent of each other. As mentioned previously, the correlation coefficient is a measure of how well the data fits a straight

line. This straight line has the form:

The y-intercept, a , and the slope, b , can be determined by using relationships found in any standard

statistical text. The parameter r^2 is often defined as the coefficient of determination, and is commonly

determined by means of the expression shown in Equation (4). The correlation coefficient, given by

Equation (2), is simply the square root of r^2 , its sign

being the same as b , the slope of the line.

This line of best fit, ordinary least squares straight line, or linear regression line, by which names it is

known, is achieved by minimizing the sum of the squared deviations. That is:

In addition, the least squares straight line has the property that the sum of the deviations is equal to zero,

that is:

Other techniques are available for evaluating regression lines other than the method of ordinary least

squares (OLS) described above. These include averages, least normal squares and reduced major axis.

An extensive treatment of each of these alternative techniques is beyond the scope of this paper, but is

given by Troutman & Williams (1987). In addition, regression analysis enables non-linear functions, such as quadratic, cubic and logarithmic

functions, to be fitted to the data. It should be noted that the largest power in the system of equations

used to determine the regression line is always twice the degree of the curve being fitted. Davis (1986)

suggested that this can be a major source of error in computer programs which fit high-order polynomials

to the data. This can result in rounding-off errors leading to unstable or unreliable solutions to the set of

simultaneous equations. Furthermore, Lumb (1974) stated that, in routine site investigations, there are

rarely sufficient data to estimate any form more complicated than a linear trend.

Mann (1987) stated that, theoretically, the linear regression model is valid only when the following

assumptions are true:

1. X and Y are measured without error;

2. X and Y are normally distributed, independent variables;
3. A linear relationship exists between X and Y ;
4. X values are linearly independent;
5. Errors of prediction are normally distributed with a mean of zero and a constant variance

(homoscedasticity);

6. Errors of prediction are serially independent (i.e. not autocorrelated - discussed briefly below, and in detail in §2.2.2).

In geotechnical engineering and the earth sciences, whilst many of these assumptions may be valid, the

first and, more importantly, the sixth assumptions are not. Matheron (1963) demonstrated that concepts

of classical statistics, such as regression analyses, are insufficient because geotechnical materials are

not serially independent. In fact, they are autocorrelated; that is, neighboring samples indicate stronger

correlation than distant samples which exhibit weak correlation (Matheron 1963, Journel & Huijbregts

1978, Li & White 1987, Jaksa 1995). This is partly explained by the randomness of the process governing

the formation of the microscopic structural units within the soil mass (Yong 1984). Rendu (1981) stated

that, since properties of geotechnical materials are rarely distributed randomly throughout a soil or rock

mass, the use of classic statistical techniques such as regression should be limited to preliminary stages

of exploration, or when the number of samples available is relatively small and the distances between

samples are large.

As a result of these limitations, regression analysis is

not used to characterize the spatial variability of geotechnical materials. Before discussing the methods of random field theory, geostatistics and fractals, it is necessary first to examine stationarity.

2.2 Random field theory

Random field theory, as it is known in geotechnical engineering (Vanmarcke 1983), is an n-dimensional extension of classical time series analysis. A time series is a chronological sequence of observations of

a particular variable usually, but not necessarily, at constant time intervals, and which may be thought

of as a one-dimensional random field. A geotechnical example of a time series is the measurement of

cone tip resistance, q_c , with depth as obtained from the cone penetration test (CPT) (Lunne et al. 1997),

as shown in Figure 1. Unlike classical statistics, time series analysis incorporates the observed behavior

that values at adjacent time steps are more related, that is autocorrelated, than those at large time steps.

When applied to the spatial variability of geotechnical materials, the time domain is replaced by the

distance domain and the data set is referred to as a random field.

2.2.1 Stationarity

The application of the techniques of random field theory and geostatistics is greatly simplified if the

data are stationary; that is, the probabilistic laws which govern the series must be independent of the

location of the samples. Data are said to be stationary, in a strict sense, if (Brockwell & Davis 1987):

- the mean, μ , is constant with distance, that is, no trend or drift exists in the data;

- the variance, σ^2 , is constant with distance (homoscedastic);
- there are no seasonal variations;
- there are no irregular fluctuations.

In practice it is usual to define stationarity in terms of second-order, or weak, stationarity. A data set is

said to satisfy second-order stationarity if its mean is constant, and its autocovariance function depends

only on the lag, which is the distance between two random variables in a random field (Chatfield 1975;

Brockwell & Davis 1987). These aspects are defined, and treated in some detail in §2.2.2.

An example of a non-stationary data set is shown in Figure 1. As can be seen, the level, indicated by

a quadratic trend, and the variance, are both a function of depth.

An important implication of the assumption of stationarity is that the statistical properties of the

random field are unaffected by a shift of the spatial origin. For example, the statistical relationships

between n samples at origin i , say $X_i, X_{i+1}, \dots, X_{i+n-1}$, are the same as the statistical relationships between n samples at origin $i+j$, say $X_{i+j}, X_{i+j+1}, \dots, X_{i+j+n-1}$ (Brockwell & Davis 1987). A stationary process is therefore invariant under spatial translation

Figure 1. An example of a non-stationary geotechnical random field.

Figure 2. Forms of non-stationarity: (a) trend non-stationarity, (b) variance non-stationarity, (c) relationship

non-stationarity. (After Bennett 1979.)

between n samples at origin $i+j$, say $X_{i+j}, X_{i+j+1}, \dots, X_{i+j+n-1}$ (Brockwell & Davis 1987). A stationary

process is therefore invariant under spatial translation

(Bennett 1979). Bennett (1979) suggested that non-stationarity falls into three separate forms, namely: trend; variance;

and relationship; each of which is shown in Figure 2. Non-stationarity of trend describes the phenomenon

that a random field is subject to a trend which may be either deterministic or stochastic; whereas non

stationarity of variance describes the pattern by which the variance is subject to change as a function of

distance. On the other hand, non-stationarity of relationship occurs when the process which controls the

random field itself, is a function of distance. Vanmarcke (1983) and Baecher & Christian (2003) stated that stationarity is equivalent to statistical

homogeneity. Baecher & Christian (2003) stated that, like most things in the earth sciences, stationarity

is an assumption that may not be true in the real world. In addition, stationarity usually depends on scale

(Fenton 1999a, Baecher & Christian 2003). Within the domain of a site, the data may appear stationary.

However, over a larger region, the data may not be stationary. Phoon et al. (2003b) proposed a formal

hypothesis test for rejecting the null hypothesis of stationarity. As the application of both random field theory and geostatistics is greatly facilitated by stationary

data, data transformation is often performed. This is the process by which a non-stationary data set is

transformed to a stationary one, and the commonly used methods are discussed below.

2.2.1.1 Data transformation

In the analysis of any random field, or time series, the first step is to plot the data and to identify any

apparent discontinuities that may exist in the series. If there are any, it is advisable to first separate

the series into a number of statistically homogeneous segments, and then to carry out the analyses on

these segments (Brockwell & Davis 1987). The stationarity test proposed by Phoon et al. (2003b) also

furnishes a systematic procedure for demarcating a data series into stationary segments. This procedure

was applied to a large CPT data set by Uzielli et al. (2005). Secondly, it is desirable to transform the data

so that they are stationary. Two techniques are commonly used to transform non-stationary data which

exhibit a trend: classical decomposition and differencing. In addition, variance transformation is used

for data which exhibit a non-stationary variance. These are treated, in turn, below.

(1) Classical decomposition

Classical decomposition assumes that a random field, X_t , is made up of a number of additive components

(Brockwell & Davis 1987):

where t_t = a slowly changing function termed the trend component; and ε_t = a random, or white noise

component.

In general, time series analysis also considers an additional component, s_t , which is due to seasonal

effects. However, since the majority of geotechnical engineering data do not contain a seasonal component

(Lumb 1974), including the data examined in the present study, no further discussion of seasonality is

given. One interesting exception is varved clay (Phoon 2006a). Such treatment, however, can be found

in a large number of time series analysis texts (e.g. Box & Jenkins 1970, Anderson 1976, Cryer 1986,

Brockwell & Davis 1987).

The aim of classical decomposition is to estimate and remove the deterministic component, μ_t , in the

hope that the residual random component, ε_t , will be stationary. This trend component can be removed

in a number of ways. In the geotechnical engineering literature the technique of OLS regression tends

to be the most widely used method for trend estimation (Lumb 1974, Alonso & Krizek 1975, Lumb

1975, Baecher 1979, Kulatilake & Southworth 1987, Spry et al. 1988, Ravi 1992), with polynomial

trends of up to order 4 being reported. Other techniques, such as smoothing by means of: a moving

average, spline fitting and exponential smoothing, have been suggested in various time series analysis

texts (Brockwell & Davis 1987).

Li (1991) correctly asserted that OLS assumes that the data are random and uncorrelated, which

is inconsistent with spatial variability analyses. Alternatively, he suggested that a technique based on

generalized least squares (GLS) be used, or the more complex methods suggested by Matheron (1973) and

Delfiner (1976). However, Kulatilake (1991) stated that, while in general agreement with Li (1991), the

GLS technique has drawbacks when applied in a practical sense. Furthermore, Ripley (1981) found that

the trend produced by GLS varied only slightly from that produced by OLS. As a result, the geotechnical

engineering community has tended to focus on the OLS method.

(2) Differencing

An alternative technique which is used extensively in time series analysis, though has found limited

use in geotechnical engineering, is that of differencing

(Chatfield 1975, Bowerman & O'Connell 1979,

Brockwell & Davis 1987). If the time series, or random field, does not contain seasonal variation,

stationarity can frequently be attained by taking the first-difference, ∇X_t , of the original data, X_t , where:

where, B , is the backshift operator given by:

In fact, any polynomial trend of degree, k , can be removed by applying the ∇^k difference operator. That

is, by starting with a model, $X_t = t^k + \varepsilon_t$, where $t^k = \sum_{j=0}^k a_j t^j$, and ε_t is stationary with zero mean, then:

is a stationary process with a mean of a_k . While differencing to any order, k , can be performed, first-, or

second-order differencing is usually sufficient (Brockwell & Davis 1987). In fact, Cryer (1986) recom

mended that care be taken to avoid over-differencing, which tends to introduce unnecessary correlations

and complicate relatively simple models. Jaksa (1995) found that the differencing method, while useful in an estimation and forecasting process,

causes the continuity of the data to be lost and, hence, is not recommended for characterizing the spatial

variability of geotechnical materials.

(3) Variance transformation

As mentioned above, the condition of stationarity is violated if the variance of the data is heteroscedastic,

that is, it varies with distance. A random field, X_t , with a non-constant variance can often be transformed

into a stationary data set, W_t , by means of either:

- Logarithmic transformation by using:
- The Box-Cox transformation by using:

Often the variance can be stabilized by choosing $\lambda = 0$

(Brockwell & Davis 1991). A number of tools are available, firstly, to ascertain whether the data are stationary, and secondly, to

enable the data to be characterized and subsequently modeled. These tools, namely the autocovariance,

autocorrelation function and cross-correlation function are treated below. Methods for determining the

stationarity of a data set are treated in §2.2.4.

2.2.2 Autocovariance and autocorrelation function

Two essential statistical properties used in time series analysis are the autocovariance, c_k , and the

autocorrelation coefficient, ρ_k , at lag, k , which are defined as:

and

where c_0 = autocovariance at lag 0; $c_k = c_{-k}$; and $\rho_k = \rho_{-k}$. The autocorrelation coefficient, ρ_k , measures the correlation between any two random field

observations separated by a lag of k units (Bowerman & O'Connell 1979). It is not possible to know c_k nor ρ_k with any certainty, but only to estimate them from samples

obtained from a population, say $X_1, X_2, \dots, X_n$. As a result, the sample autocovariance, c^*k , and the

sample autocorrelation coefficient, at lag k , r_k , are defined, respectively, as:

and

where $\bar{X}$ = average of the observations $X_1, X_2, \dots, X_n$; and $0 \leq k < n$. The sample autocovariance function (ACVF), or autocovariogram, is the plot, or graph, of c^*k , for

lags $k = 0, 1, 2, \dots$. The sample autocorrelation function (ACF), or correlogram, is the graph of r_k for

lags $k = 0, 1, 2, \dots, K$; where K is the maximum number of lags that r_k should not be calculated beyond.

While the ACF can be evaluated for all lags up to $n-1$, it

is not advisable since, as k tends toward n , the number of pairs reduces and, as a consequence, the reliability of the estimate r_k of the true autocorrelation function, ρ_k , also decreases. Various authors have suggested the following values for K :

- $K = n/4$ (Box & Jenkins 1970, Chatfield 1975, Anderson 1976, Pankratz 1983, Davis 1986);
- $K = 12$ for non-seasonal time series (Bowerman & O'Connell 1979);
- $K = \sqrt{n} + 10$ (Cryer 1986);
- $20 \leq K \leq 40$ (Brockwell & Davis 1987, Hyndman 1990).

As one might expect, the accuracy of the sample autocorrelation and autocovariance functions are

directly related to the number of observations, n , in the random field. Little guidance is given regarding

the minimum number of observations, though Box & Jenkins (1970), Anderson (1976), and Davis (1986)

recommended that n be greater than 50. With particular reference to the spatial variability of soils, Lumb

(1975) suggested that, for a full three-dimensional analysis, the minimum number of test results needed

to give reasonably precise estimates is of the order of 10^4 , which is prohibitively large, even for a special

research project. On the other hand, Lumb (1975) recommended, that the best that can be achieved in

practice, is to study the one-dimensional variability, either vertically or laterally, using n of the order of

20 to 100.

The ACF is used widely throughout the geotechnical literature, whereas the use of the autocovariance

function is more limited. The ACF allows the scale of fluctuation or correlation distance to be quanti

fied. Vanmarcke (1977) stated that, in order to model the spatial variability of geotechnical materials, a

minimum of 3 parameters is needed: (1) the mean, μ ; (2) a measure of the variance, σ^2 (e.g. standard

deviation, σ , or coefficient of variation); and (3) the scale of fluctuation, θ , which expresses the corre

lation of properties with distance. A large value of θ , for a particular soil property, implies the property

fluctuates with distance slowly about the mean, suggesting a more continuous deposit, whereas a small

θ implies the property fluctuates rapidly about the mean, suggesting a more randomly varying material.

A purely random data set, or white noise, is characterized by an ACF with the following properties:

In addition, several authors (Box & Jenkins 1970, Chatfield 1975, Bennett 1979, Bowerman &

O'Connell 1979, Davis 1986) have suggested that the stationarity of a random field can be determined

from an inspection of the ACF. These authors stated that non-stationarity is punctuated by an ACF which

slowly dies-down over large lags. Conversely, ACFs which die-down rapidly are thought to represent

stationary random fields. While these rules are somewhat subjective, it is difficult to define quantitative

limits (e.g. the maximum number of lags beyond which the autocorrelations should essentially be equal

to zero), as these are very much dependent on the scale of the problem. A number of other techniques

for the evaluation of stationarity are available, and these will be examined later in this section.

The theoretical ACF of a stationary random field tends either to cut-off after a particular lag $k = q$, or

die-down with increasing lag, k (Bowerman and O'Connell 1979). Since r_k is an estimation of ρ_k , it is

difficult to know, with certainty, when $\rho_k = 0$. Many authors, however, have provided some guidance to

the calculation of q , that is, the lag number at which the theoretical ACF is thought to be equal to zero.

That is, $\rho_k = 0$ for $k > q$, if:

- $|\rho_k| = 2 \sqrt{n} (1 + 2^{k-1} \sum_{i=1}^k r_i^2)^{1/2} \approx \pm 1.96 \sqrt{n}$, often referred to as Bartlett's approximation, which corresponds

to two standard errors of the estimates (Box & Jenkins 1970, Chatfield 1975, Anderson 1976,

Brockwell & Davis 1987, Hyndman 1990);

- $|\rho_k| = 2 \sqrt{n+1} (1 + 2^q \sum_{j=1}^q r_j^2)^{1/2}$ for $k > q$; a rough rule of thumb suggested by Bowerman and O'Connell (1979).

Of these methods, the former appears to be the most widely used throughout the time series litera

ture. Jaksa et al. (2000) provided a straightforward method for implementing Bartlett's approximation,

and this method is explained, by means of an example, later in the paper. Phoon (2006b) proposed

a new promising technique based on bootstrapping that would allow the uncertainties in the sample

autocorrelation function to be quantified at all lags.
 Table 1. Theoretical autocorrelation functions used to determine the scale of fluctuation, θ . (After Vanmarcke 1977, 1983, Li and White 1987). Autocorrelation function
 Model Scale of fluctuation, θ $\rho_{\theta z} = \begin{cases} 1 - |\theta z|/a & \text{for } |\theta z| \leq a \\ 0 & \text{for } |\theta z| \geq a \end{cases}$ Triangular model a $\rho_{\theta z} = e^{-|\theta z|/b}$ Simple exponential $2b$ $\rho_{\theta z} = e^{-(|\theta z|/c)^2}$ Squared exponential, or $\sqrt{\pi c}$ Gaussian, model $\rho_{\theta z} = e^{-|\theta z|/d (1 + |\theta z|/d)}$ Second-order, autoregressive $4d$ process $\rho_{\theta z} = e^{-|\theta z|/\alpha} \cos(\theta z/\alpha)$ Cosine exponential α

2.2.3 Evaluating the scale of fluctuation

As mentioned above, in order to model the spatial variability of geotechnical materials, a minimum of

3 parameters is needed: (1) the mean, μ ; (2) a measure of the variance, σ^2 (e.g. standard deviation,

σ , or coefficient of variation); and (3) the scale of fluctuation, θ . Quantifying the first two of these is

relatively straightforward using standard procedures given in all basic statistical texts (e.g. Smith 1986,

Davis 1986). In order to quantify the scale of fluctuation, Vanmarcke (1983) suggested the following

techniques: (1) the variance function method; (2) the autocorrelation method; and (3) the mean crossings

approximation. Of these, most practitioners make use of the second method, which involves fitting a

theoretical autocorrelation function, as shown in Table 1, to the sample ACF using standard curve fitting

techniques such as ordinary least squares regression. Jaksa et al. (2000) proposed an alternative method based on Bartlett's approximation. This method

will be described later, when examining the data pertaining to Keswick Clay.

2.2.4 Assessing stationarity

Bennett (1979) proposed five methods for assessing the stationarity of data: (1) eyeballing the scatterplot;

(2) examination of histogram plots; (3) inspection of the ACF; (4) the number of significant trends; and

(5) significance tests on trends. Of these, the third method is the most commonly adopted, but it suffers

from the limitation that it is somewhat subjective. Alonso & Krizek (1975) proposed the use of the

nonparametric, statistical runs test. However, Jaksa (1995) found that this test was a poor indicator of

stationarity. Ravi (1992) proposed the use of Kendall's τ test and Jaksa (1995) found this method to be

quite robust, although it is generally accepted that no

method is entirely suitable on its own. The test

essentially involves the calculation of the test statistic, τ , which measures the probability of concordance

minus the probability of discordance (Daniel 1990). The observation pairs, (X_i, Y_i) and (X_j, Y_j) , are

said to be concordant if the difference between X_i and X_j is in the same direction as the difference between

Y_i and Y_j . That is, if either $X_i > X_j$ and $Y_i > Y_j$, or $X_i < X_j$ and $Y_i < Y_j$, then the pairs are concordant.

Discordance, on the other hand, is when the directions of the differences are not the same. If $X_i = X_j$

and/or $Y_i = Y_j$, then the pairs are neither concordant nor discordant. The test statistic, τ , is given by

(Daniel 1990, Noether 1991):

where n = number of pairs of (X, Y) observations. To obtain S the following process is used:

1. The observations (X_i, Y_i) are ranked in a column according to the magnitude of the X s, with the smallest X first, through to the largest X last. The X s are thus arranged in their natural order.
2. Each Y value is then successively compared with each Y value appearing below it. In making these comparisons, a pair of Y values, that is, the value of Y in question and the value of Y below it, is said to be in natural order if the Y below is larger than the Y above. On the other hand, a pair of Y values is in reverse natural order if the Y below is smaller than the Y above.
3. The parameter, P , is the number of pairs in natural order, and the parameter, Q , is the number of pairs in reverse natural order.
4. S is then simply the difference between P and Q ; that is, $S = P - Q$.

The value of Kendall's τ lies between +1 and -1 with: $\tau = +1$ indicating perfect positive correlation;

$\tau = -1$ indicating perfect negative correlation; and $\tau = 0$ indicating no correlation. Hence, when applied

to the assessment of stationarity, a value of τ close to zero indicates that the data are stationary, whereas a

value of τ near +1 or -1 indicates non-stationary data. Usually, for some confidence level specified by

the user, the value of τ is compared with critical values of τ tabulated in many statistics texts (e.g. Daniel

1990, Noether 1991). Tables of critical τ values are rarely given for data sets where n exceeds about 40.

For large values of n , Daniel (1990) recommended the use of the following equation:

The parameter $z\tau$ is normally distributed with a mean of zero and variance of unity and, as a conse

quence, the value of $z\tau$ can be compared with values given in standard, normal distribution tables. Hence,

if $z\tau \leq \pm 1.96$ then the data can be assumed to be stationary with 95% confidence. Additionally, Noether

(1991) suggested the use of the parameter, c , given by:

The parameter, c , suggests that the probability of concordance, P , is c times more likely than the

probability of discordance, Q . Kendall's τ test will be adopted later when the Keswick Clay data are

examined in §3.

Phoon et al. (2003b) proposed a modified Bartlett statistical test to provide a more rational basis for

assessing stationarity. The authors proposed a new test statistic, $B_{\max}$, which is obtained by computing

a continuous Bartlett statistic profile using a moving sampling window and identifying the maximum

value. The authors' test statistic is more discriminating than other traditional classical test statistics

as it incorporates the appropriate correlation structure of the underlying data and serves as a useful

complement to the existing statistical methods. Details of the method are given by Phoon et al. (2003b).

2.3 Geostatistics

The mathematical technique, which is now universally known as geostatistics, was developed to assist

in the estimation of changes in ore grade within a mine, and is largely a result of the work of Krige

(1951) and Matheron (1965). Since its development in the early 1960s, geostatistics has been applied to

many disciplines including: groundwater hydrology and hydrogeology; surface hydrology; earthquake

engineering and seismology; pollution control; geochemical exploration; and geotechnical engineering.

In fact, geostatistics can be applied to any natural phenomena that are spatially or temporally associated

(Journel & Huijbregts 1978, Hohn 1988).

As mentioned previously, geostatistics is based on the regionalized variable, that is, one that can be

represented by random functions, rather than the classical approach which treats samples as independent

random variables. The regionalized variable has properties that are partly random and partly spatial and

has continuity from point to point; however, the changes are so complex that they cannot be described

by a tractable deterministic function (Davis 1986).

One of the basic statistical measures of geostatistics is the semivariogram, which is used to express

the rate of change of a regionalized variable along a specific orientation. The semivariogram is treated

below.

2.3.1.1 Semivariogram

The semivariogram is a measure of the degree of spatial

dependence between samples along a specific

orientation, and represents the degree of continuity of the property in question. The semivariogram, $\gamma(h)$,

is defined by the following equation:

where: X_i = value of the property, X , at location, i ; X_{i+h} = value of the property, X , at location, $i+h$;

h = displacement between the data pairs; and $E[\dots]$ = expected value.

Thus, the semivariogram is defined as half the expected value, or mean, of the squared difference

between pairs of points, X_i and X_{i+h} , separated by a displacement, h . If the regionalized variable is 0 10 20 30 40 50 60 70 80 -0.2 0 0.2 0.4 0.6 0.8 1 0 20 40 60 80 10 30 50 70 Autocorrelation, $\rho(h)$ Semivariogram Displacement, h Semivariogram, $\gamma(h)$ Autocorrelation

Figure 3. Relationship between the semivariogram, $\gamma(h)$, and the autocorrelation, $\rho(h)$, for a stationary regionalized

variable. (After Davis 1986.)

stationary and normalized to have a mean of zero and a variance of 1.0, the semivariogram is the

mirror image of the ACF, as shown in Figure 3. Fenton (1999a) stated that the semivariogram gives

essentially the same information as the ACF because, for stationary processes, they are related according

to Equation (22). An advantage of the semivariogram, in comparison to the ACF, is that it does not depend

on the mean. Fenton (1999a) noted that this is particularly useful since many of the problems associated

with the ACF relate to this dependence. Even though a regionalized variable is spatially continuous, like the ACF, it is not possible to know

its value at all locations. Instead its values can only be determined from samples taken from a popula

tion. Thus, in practice, the semivariogram must be estimated from the available data, and is generally

determined by the following relationship:

where $\gamma * h$ = experimental semivariogram (i.e. one based on the sampled data set); and N = number of

data pairs separated by the displacement, h . Clark (1980) stated that the experimental semivariogram is theoretically restricted to distances of

one-quarter of the total extent of the sampled information, however, in practice, one-half is generally

used (Journel & Huijbregts 1978, Clark 1979, 1980). (For example, if a CPT sounding is performed to a

depth of 5,000 mm, its corresponding semivariogram would normally be calculated for values of h from

0 to 2,500 mm). In addition, the accuracy of $\gamma * h$ is directly related to the number of data pairs, N (Brooker

1991), with the minimum number of pairs needed for a reliable estimate of $\gamma * h$ being between 30 and 50

(Journel & Huijbregts 1978); with some authors suggesting as many as 400 to 500 (Clark 1980). The strength of geostatistics is that it provides, through the semivariogram, a framework for the esti

mation of variables. In fact, it can be shown that geostatistics provides the best, linear, unbiased estimator

(BLUE) (Journel & Huijbregts 1978, Clark 1979, Rendu 1981). Whilst the experimental semivariogram

is known only at discrete points, the estimation procedure, known as kriging, requires the semivariogram

values be known for all h . Thus, it is necessary to model the experimental semivariogram, $\gamma * h$, as a con

tinuous function, γh . Table 2 and Figure 4 present a number of semivariogram models that are commonly

used in the literature, the most widely applied of these being the spherical model. With particular reference to the spherical model, three parameters are used (Fig. 5):

C_0 - is defined as the nugget effect and arises from the regionalized variable being so erratic over a short distance that the semivariogram goes from zero to the level of the nugget in a distance less

Table 2. Common semivariogram models.

Model	Mathematical function	Remarks
Pure Nugget	$\gamma(h) = C_0$	C_0 = Nugget
Spherical	$\gamma(h) = C \left(\frac{3h}{2a} - \frac{h^3}{2a^3} \right) + C_0$ when $h \leq a$ $\gamma(h) = C + C_0$ when $h \geq a$	$C + C_0$ = Sill; Range = a
Exponential	$\gamma(h) = C(1 - e^{-h/a}) + C_0$	Effective range = $3a$
Gaussian	$\gamma(h) = C(1 - e^{-h^2/a^2}) + C_0$	Effective range = $\sqrt{3}a$
Linear	$\gamma(h) = ph + C_0$	p = slope
Power	$\gamma(h) = ph^\alpha + C_0$	$0 < \alpha < 2$
de Wijsian	$\gamma(h) = 3\alpha \ln(h) + C_0$	α = absolute dispersion

In addition, each of the models, above, obey $\gamma(0) = 0$; * :
 From Olea (1991).
 0 50 100 0 50 100 Spherical Gaussian
 Exponential Displacement, h 0 50 100 0 50 100 Power de
 Wijsian Linear Displacement, h S e m i v a r i o g r a m ,
 $\gamma(h)$ S e m i v a r i o g r a m , $\gamma(h)$

Figure 4. Common model semivariograms, with C_0 set to zero. Displacement, h Range, a C_0 C S e m i v a r i o g r a m , $\gamma(h)$

Figure 5. Parameters defining the spherical model. than the sampling interval. The nugget effect is the result of three separate phenomena (Rendu 1981):

1. microstructures within the geological material - which have been observed from the study of the

spatial variability of mineral concentrations of core samples. Two adjacent cores will exhibit a nugget

effect when one of them contains a nugget and the other does not (Journel & Huijbregts 1978). Several researchers have stated that soils also exhibit this behavior (Calle et al. 1987, Li & White 1987, Soulié et al. 1990, Jaksa 1995);

2. sampling, or statistical, errors - $C(0)$ depends greatly on the spacing between individual samples (Brooker 1977, Journel & Huijbregts 1978, Clark 1979, 1980, de Marsily 1982);

3. measurement errors - if it were possible to obtain a repeat measurement at precisely the same location, the observations would differ by an amount directly dependent on the measurement technique. Measurement errors are also manifested by a non-zero value of the semivariogram at zero lag. It has been standard practice in the treatment of the spatial variability of soils to attribute the nugget effect solely to measurement errors associated with the testing method (Baecher 1986, Wu & El-Jandali 1985, Filippas et al. 1988, Phoon & Kulhawy 1999). However, Li & White (1987) suggested that while some of the nugget effect can be attributed to measurement errors, the possibility of small-scale random effects, that is, microstructures, cannot be ignored in geotechnical properties; $C + C(0)$ - is known as the sill which measures half the maximum, on average, squared difference between data pairs; and a - is defined as the range of influence, or range, and is the distance at which samples become independent of one another. Data pairs separated by distances up to a are correlated, but not beyond. Clark (1979) described the process of fitting a model to an experimental semivariogram as essentially

a trial-and-error approach, usually achieved by eye. Brooker (1991) suggested the following technique

as a first approximation in determining the appropriate parameters for a spherical model:

- The experimental semivariogram and variance of the data are plotted;
- The value of the sill, $C + C(0)$, is approximately equal to the variance of the data;
- A line is then drawn with the slope of the semivariogram near the origin, which intersects the sill at two thirds the range, a ;
- This line intersects the ordinate at the value of the nugget effect, $C(0)$. In addition, Brooker (1991) stated that the accuracy of the modeling process depends on both, the

number of pairs in the calculation of the experimental semivariogram, and the lag distance at which it is

evaluated. Journel & Huijbregts (1978) suggested that automatic fitting of models to experimental semivari

ograms, such as least squares methods, should be avoided. This is because each estimator point, $\gamma * h$, of

an experimental semivariogram is subject to an estimation error and fluctuation which is related to the

number of data pairs associated with that point. Since the number of pairs varies for each point so, too,

does the estimation error. The authors recommended that the weighting applied to each estimated point,

$\gamma * h$, should come from a critical appraisal of the data, and from practical experience. Similar to the ACF of random variables, the semivariogram is greatly facilitated by stationarity, that

is, the semivariogram depends only on the separation distance and not on the locality of the data pairs.

The regionalized variable can be regarded as consisting of two components: the residual and the drift.

If a drift, or trend, exists in the data, which leads to non-stationarity, it must first be removed (Journel

& Huijbregts 1978, Henley 1981). It has been shown by Davis (1986), that if the drift is subtracted

from the regionalized variable, the residuals will themselves be a regionalized variable and will have

local mean values of zero. In other words, the residuals will be stationary and the semivariogram can be

evaluated.

2.4 Fractals

Fenton (1999a) applied the concept of fractals to geotechnical engineering. According to its founder,

Mandelbrot (1982), a fractal is a rough or fragmented geometric shape that can be subdivided in parts,

each of which is (at least approximately) a reduced/size copy of the whole. With respect to geotechnical

properties, the characteristics are the same, or self-similar behavior, irrespective of the observation

scale. Fenton (1999b) suggested that such fractal or long memory behavior is likely to be present due

to the large-scale mixing processes, such as erosion, transportation, deposition and weathering that are

involved in the formation of soils. Jaksa & Fenton (2002) examined 448 CPT data sets, both vertical and horizontal and concluded that

the self-similar behavior was apparent in only a few of the data sets. As a result, the vast majority of

geotechnical data can be adequately modeled using random field theory and geostatistics. Consequently,

no further treatment of fractals is given here. However, the interested reader is directed to the papers

cited above for further information on fractals and their application to geotechnical engineering data.

3 CASE STUDY: KESWICK CLAY

This section demonstrates the application of the techniques described above using CPT data to charac

terize the small-scale spatial variability and triaxial test data to quantify the large-scale variability. The

CPT and triaxial data were obtained from tests performed within, and samples obtained from, a stiff to

hard, fissured, over-consolidated clay of high plasticity, known as Keswick Clay. Much of the city of

Adelaide, South Australia, is underlain by this clay, the properties of which have been demonstrated by

Cox (1970) to be remarkably similar to those of the ubiquitous London Clay. The following sub-sections

describe the data and small and large-scale characterization

of the Keswick Clay.

3.1 Small-scale characterization

In order to characterize the small-scale spatial variability of a geotechnical material it is necessary to

obtain a very large amount of closely-spaced and accurate data. As mentioned above, a minimum of 50

data points are needed, however, it is desirable to obtain many more. The CPT is ideal for such purposes

as it yields a great number of closely-spaced data which, in the majority of systems, is spaced at an

interval of 20 mm. In addition, it has been shown by Orchant et al. (1988) and Jaksa et al. (1997b) that

the CPT yields measurements with minimal random measurement error (i.e. of the order of 2 to 5%).

Furthermore, the continuous and electronic nature of the CPT lends itself to computer data acquisition,

which again greatly facilitates recording large numbers of measurements. The following sub-section

describes the cone penetration testing program adopted to obtain the data needed for subsequent spatial

variability analyses.

3.1.1 Overview of CPT data

The data examined in this section were derived from a field investigation performed in the Adelaide

central business district. Measurements from several CPTs from two sites are investigated, namely the

South Parklands and the Keswick sites. A total of 224 vertical CPTs were performed at the South

Parklands site in a square grid arrangement, as shown in Figure 6, to a typical depth of 5 m, using

truck-mounted drilling rigs. As indicated in Figure 6, the CPTs were performed at horizontal spacings

of 0.5, 1 and 5 m. In order to limit other influences, such as climatic, operator and equipment, the CPTs

were performed in as short a time period as possible (approx. 43 days), and a single cone penetrometer

and operator was used.

Figure 6. Layout of CPTs at the South parklands site. A single, horizontal CPT was carried out at the Keswick site, as shown in Figure 7, to a penetration

distance of 7.6 m, into an existing railway embankment using a trailer mounted rig. At both sites, the CPTs

were performed in accordance with the international standard (ISSMFE 1989). The CPT measurements

were obtained at penetration intervals of 5 mm, using a computer-based, data acquisition system, which

is described in some detail by Jaksa & Kaggwa (1994). Further details regarding the data and testing are

given by Jaksa (1995). In addition, the complete set of CPT data can be obtained from the author's web

page (see Jaksa 1995).

3.1.2 Characterization of vertical variability

In order to illustrate the application of the techniques described above, a typical vertical CPT, C8 (Fig. 5),

is examined in detail. In the analyses that follow, only cone tip resistance, q_c , measurements are examined.

Hence, measurements of q_c against depth below ground are shown in Figure 7 for CPT C8. The spatial variability analyses performed on the CPT data that follow were achieved using soft

ware, known as SemiAuto, developed specifically for this purpose. This Windows-based, PC program is

described in detail by Jaksa (1995). As mentioned in §2.2.1, in order to characterize the spatial variability of a data set, it is desirable that

it be statistically homogeneous. In the case of the CPTs

performed at the South Parklands site, Keswick

Clay is overlain by 3 soil layers of limited importance and interest. Hence, in order to characterize the

spatial variability of Keswick Clay, it is first necessary to remove measurements pertaining to the other

layers. As shown in Figure 6, continuous core samples indicated that, for C8, Keswick Clay is encountered

at a depth of 1.1 m below the ground surface, which is indicated in Figure 8. As a result, measurements

above 1.1 m are removed from the data set, as shown in Figure 9.

Figure 7. Cross-section of embankment and extent of the horizontal CPT at the Keswick site. 6000 5000 4000 3000 2000 1000 0 0.5 1.0 1.5 2.0 2.5 Cone Tip Resistance, q_c (MPa) Keswick Clay Depth Below Ground (mm)

Figure 8. Measured cone tip resistance, q_c , for

CPT C8. 5500 5000 4500 4000 3500 3000 2500 2000 1500 1000 0 0.5 1.0 1.5 2.0 2.5 Cone Tip Resistance, q_c (MPa) Quadratic Trend $r^2 = 0.719$ Depth Below Ground (mm) Figure 9. Measured cone tip resistance, q_c , for CPT C8, within Keswick Clay.

The next step is to assess the stationarity of the data set in Figure 9. As shown, a strong OLS quadratic

trend is present, with a coefficient of determination, $r^2 = 0.719$. Kendall's τ test confirms data non

stationarity ($S = 76,199$; $\tau = 0.270$; $c = 1.056$ and $z\tau = 1.973 \approx \text{Fail}$).

In order to achieve stationarity, the quadratic trend is removed, as shown by Equation (24), where the

expression between parentheses represents the OLS quadratic trend, and Rq_c is the remainder, or residual

as it is known.

Fenton (1999a) recommended that it is best to assume that the data are stationary, rather than remove

some low-order trend, unless it is known that the trend is due to some physical basis that is likely to be

repeated at other sites. In the present case study, Keswick Clay is known to be over-consolidated as a

result of desiccation (Stapledon 1970) and such soils generally exhibit a quadratic variation of undrained

shear strength with depth and, as a consequence, it is appropriate to remove such a trend.

It is apparent from the residuals, shown in Figure 10 and obtained from Equation (24), that the data are

stationary since there is no apparent trend with depth and there are no obvious indications that variance

is a function of depth. Kendall's τ test, confirms data stationarity ($S = 13,175$; $\tau = 0.044$; $c = 1.091$ and

$z \tau = 1.820 \approx \text{Pass}$). It is worth noting, at this point, that by removing an n th order trend, lower order

trends, i.e. $n-1$, $n-2$, ..., 1 , are also removed. Hence, by removing the quadratic trend in this case, a

linear drift is also removed, such that the remaining residuals have a mean of zero.

3.1.2.1 Random field theory

The sample ACF can then be determined by substituting the residuals (Fig. 9) into Equation (16), as

shown in Figure 11. Superimposed on the sample ACF in Figure 11, are two of the more commonly

applied models suggested by Vanmarcke (1977, 1983), as given previously in Table 1.

The models are fitted using the method of OLS, from which the following parameters were obtained:

- Model 2: $p(y) = e^{-|y|/b}$, where: $b = 91.4$ mm; and
- Model 3: $p(y) = e^{-(|y|/c)^2}$, where: $c = 96.7$ mm.

The scale of fluctuation, θ_V , is then calculated, for each model, by substituting each parameter into

the relevant relationship, as given in Table 1. That is:

- Model 2: $\theta \vee 2 = 2b = 182.7 \text{ mm}$; and
- Model 3: $\theta \vee 3 = \sqrt{\pi c} = 171.4 \text{ mm}$.

5500

5000

4500

4000

3500

3000

2500

2000

1500

1000 -0.8 -0.6 -0.4 -0.2 0 0.2 0.4 0.6 Cone Tip Resistance,
q_c (MPa)

D

e p

t h

B

e l

o w

G

r o

u n

d

(m

m)

Figure 10. Residuals of q_c , for CPT C8, after removal

of the quadratic trend. -1 -0.8 -0.6 -0.4 -0.2 0 0.2 0.4
 0.6 0.8 1 500 1000 Sample ACF Model 3 Model 2 Distance, y
 (mm) A u t o c o r r e l a t i o n , r_y Figure 11. Sample
 and model ACFs obtained from the residuals of q_c for C8.
 -1 -0.8 -0.6 -0.4 -0.2 0 0.2 0.4 0.6 0.8 1 500 1000 Sample
 ACF Bartlett's Limits Enlargement, shown right Distance, y
 (mm) -0.2 -0.1 0 0.1 0.2 150 200 250 300 350 400 Sample ACF
 Bartlett's Limits ENLARGEMENT of that shown left Distance,
 y (mm) A u t o c o r r e l a t i o n , r_y A u t o c o r r e l a t i o n , r_y

Figure 12. Sample ACF, showing Bartlett's limits, obtained from the residuals of q_c for CPT C8, used to evaluate

Bartlett's distance. 0 50 100 150 200 250 300 0 50 100 150
 200 250 300 Equality Line S c a l e o f F l u c t u a t i o n , θ_v (mm) OLS Line of Best Fit Bartlett's Distance, r_B (mm)

Figure 13. Relationship between the scale of fluctuation, θ , and Bartlett's distance, r_B .

In addition, Bartlett's limits are determined as:

- Bartlett's limit $\pm 1.96/\sqrt{n} = \pm 1.96/\sqrt{769} = \pm 0.071$. By superimposing Bartlett's limits onto the sample ACF, the distance termed Bartlett's distance, r_B ,

over which the samples are autocorrelated, i.e. exhibit strong correlation, is determined by identifying

the distance at which the sample ACF first crosses Bartlett's limits, as shown in Figure 12. It is evident

from Figure 12 that the sample ACF intersects Bartlett's limits at a distance of approximately 240 mm.

Hence $r_B = 240$ mm, which is consistent with the values of θ_v evaluated above. By examining the data from 30 CPTs, Jaksa et al. (2000) found a strong relationship ($r^2 = 0.89$)

between θ_v and r_B , as shown in Figure 13 and Equation (25). As shown by Figure 13 and Equation (25),

the relationship is essentially one of equality, i.e. $\theta = r_B$. This is useful, since r_B is much easier to evaluate

than θ .

where θ V and r B , are both expressed in millimeters. θ
 0.005 0.010 0.015 0.020 0.025 0.030 0 200 400 600 800 1000
 1200 1400 1600 1800 2000 Experimental Semivariogram
 Spherical Model Distance, y (mm) S e m i v a r i o g r a m
 , γ y (M P a ²)

Figure 14. Experimental and model semivariograms of residual q c data from C8.

3.1.2.2 Geostatistics

In order to evaluate the experimental semivariogram, the residuals, shown previously in Figure 10, are

substituted into Equation (23). The resulting experimental semivariogram is shown in Figure 14, along

with a spherical model semivariogram that was fitted by eye. The spherical model is expressed by:

where $a = 330$ mm; $C = 0.0231$ MPa² ; and $C_0 = 0.0018$ MPa² .

As can be seen clearly from Figure 14, there is justification for inclusion of a nugget, C_0 , within the

spherical model. As discussed earlier, C_0 accounts for microstructures within the geological material,

sampling or statistical errors, and random measurement errors.

3.1.3 Characterization of horizontal variability

It is possible to assess horizontal spatial variability by examining measurements obtained from tests per

formed at various horizontal intervals, which are usually consistent, at a constant depth below the ground.

Due to the reasonably large number of closely spaced vertical CPTs, as shown in Figure 6, it is possible to

assess the horizontal spatial variability of Keswick Clay. Analyses performed by Jaksa (1995) indicated

that a horizontal spacing of 1 m yielded inaccurate characterization of the horizontal spatial variability.

Hence, it was decided to conduct an additional 50 CPTs (CD1 to CD50) spaced at 0.5 m horizontal

intervals, as shown in Figure 6.

The data for the horizontal spatial variability analyses were obtained by establishing a spreadsheet with

q c measurements from adjacent CPTs were placed in neighboring columns, with each row representing

a consistent depth below ground at vertical increments of 5 mm. A series of horizontal transects were

then generated from each of the rows. Four such horizontal transects are summarized in Figure 15, for

depths 3.5, 4.0, 4.5 and 5.0 m below ground.

Whilst these data appear to be non-stationary, Fenton (1999a) suggested that this may not be the case

when large domain sizes are considered. In consequence, the data shown in Figure 15 are examined

without de-trending.

The sample ACFs for these four horizontal transects are shown in Figure 16 and were obtained using

Equation (16). It is evident from this plot that the horizontal scale of fluctuation, θ_H , varies between 2.8

and 4.4 m, with an average of approximately 3.5 m.

The experimental semivariograms of the four horizontal transects are shown in Figure 17 and were

obtained using Equation (23). It is evident from this plot that the horizontal range of influence, a_H , is of

the order of 10 m. Both θ_H and a_H are consistent with those reported in the literature (Jaksa 1995).
 0 0.5 1.0 1.5 2.0 2.5 3.0 3.5 0 2.5 5 7.5 10 12.5 15 17.5 20 22.5 25
 3.5 m below ground 4.0 m below ground 4.5 m below ground
 5.0 m below ground Lateral Distance, x (m) CD1 CD50 C o n e
 T i p R e s i s t a n c e , q c (k P a)

Figure 15. Horizontal spatial variability data along

transect CD1 to CD50. Horizontal Distance, x (m) -1 -0.8
-0.6 -0.4 -0.2 0 1 2 3 4 5 6 0.2 0.4 0.6 0.8 1 Bartlett's
Limits 3.5 m below ground 4.5 m below ground 4.0 m below
ground 5.0 m below ground A u t o c o r r e l a t i o n , r_x

Figure 16. Sample autocorrelation functions of the horizontal spatial variability data for transect CD1 to CD50. Given the limited number of data contributing to the sample ACFs and experimental semi-variograms

given in Figures 16 and 17, respectively, a horizontal CPT was performed into the side of an embankment

in Keswick Clay at Keswick, as discussed previously in §3.1 and shown in Figure 7. The benefit of such

a test is the close spacing of the data (5 mm cf 500 mm for CPTs CD1 to CD50) and, hence, the large

data set (1,524 cf 50 for CPTs CD1 to CD50). The q_c data for the horizontal CPT at the Keswick site is shown in Figure 18. As shown, the first

2 m of data have been ignored, as these may be affected by seasonal and surface vegetation influences,

which may detract from the characterization of the horizontal spatial variability of the soil. Again, the

sample ACF and experimental semivariogram are evaluated as before and are shown in Figures 19 and

20, respectively. The resulting horizontal scale of fluctuation, θ_H , is approximately 1.05 m and the range, a_H , is around

200 mm. These values are somewhat inconsistent and their relationship differs from that obtained pre

viously. (The relationship between θ and a is examined and discussed later in §3.3.) This appears to be

related to the manner in which the sample ACF decays with increasing lag relative to Bartlett's limits, 0.00 0 2 4 6 8 10 12 14 0.05 0.10 0.15 0.20 0.25 3.5 m below ground 4.5 m below ground 4.0 m below ground 5.0 m below ground Horizontal Distance, x (m) S e m i v a r i o g r a m , γ_x (M P a 2)

Figure 17. Experimental semivariograms of the horizontal spatial variability data for transect CD1 to CD50. 0 0.5 1.0 1.5 2.0 2.5 3.0 3.5 1000 2000 3000 4000 5000 6000 7000 8000 Horizontal Penetration Distance, x (mm) Cone Tip Resistance, q_c (MPa)

Figure 18. Data from horizontal CPT at Keswick site. -1 -0.8 -0.6 -0.4 -0.2 0 0.2 0.4 0.6 0.8 1 500 1000 1500 Horizontal Distance, x (mm) Autocorrelation, r_x

Figure 19. Sample autocorrelation function of horizontal CPT from Keswick site. 0.00 0.02 0.04 0.06 0.08 0.10 0.12 0.14 0 500 1000 1500 2000 2500 3000 Experimental Semivariogram Spherical Model Horizontal Distance, x (mm) Semivariogram, γ_x (kPa²)

Figure 20. Experimental semivariogram of horizontal CPT from Keswick site. -1 -0.8 -0.6 -0.4 -0.2 0 0.2 0.4 0.6 0.8 1 500 Horizontal Distance, x (mm) Autocorrelation, r_x

Figure 21. Sample autocorrelation function of horizontal CPT from Keswick site (4 to 6 m).

as shown in Figure 19. In an effort to examine this further, the horizontal CPT data from the Keswick

site (Fig. 18) is sub-sampled to produce a smaller data set, i.e. between 4 and 6 m. The resulting sample ACF and experimental semivariogram are shown in Figures 21 and 22, respec

tively. It is evident from these plots that θ_H is approximately 100 mm and a_H is around 180 mm. The

relationship between these values is more consistent with that expected.

3.1.3.1 Discussion of horizontal spatial variability results

The results of the horizontal spatial variability analyses performed on the vertical CPTs from the South

Parklands site appear to be inconsistent with those obtained from the single horizontal CPT performed

at the Keswick site. In the former case, θ_H was found to be approximately 3.5 m, while a_H was found to

be of the order of 10 m. On the other hand, the horizontal CPT yielded $\theta_H \approx 100$ mm and $a_H \approx 180$ mm,

as discussed above. By the very nature of the CPT, one would expect soils to exhibit correlation over some finite distance.

This is due to the fact that q_c is a measure of strength, spatially averaged within a plastically failed

region, as shown in Figure 22. Hence, one would expect the soil to exhibit correlation within this failed

zone. Teh & Houlsby (1991), using a strain path approach, quantified the extent of this failure zone by $\theta_{0.00} \theta_{0.02} \theta_{0.04} \theta_{0.06} \theta_{0.08} \theta_{0.10} \theta_{0.12} \theta$ 100 200 300 400 500 600 700 800 900 1000 Experimental Semivariogram Spherical Model Horizontal Distance, x (mm) Semivariogram, $\gamma(x)$ (kPa²)

Figure 22. Experimental semivariogram of horizontal CPT from Keswick site (4 to 6 m). Plastic Soil Elastic Soil $a_c = 17.9$ mm $r_p = 120$ mm $z_p = 65$ mm $\beta = 60^\circ$

Figure 23. Extent of the failure zone in Keswick Clay from the CPT.

the use of two parameters: r_p – the radial distance of the plastic boundary from the axis of penetration

measured at a large enough distance above the cone tip; and z_p – the distance between the cone tip and the

plastic boundary measured along the axis of the penetrometer (Fig. 23). Teh & Houlsby (1991) expressed

the parameters r_p and z_p as a function of a_c , β and I_r – the radius of the cone penetrometer, the apex

angle of the cone penetrometer tip, and the rigidity index of the soil, respectively (Fig. 23). For standard

cones, including the one adopted in the present case study, $a_c = 17.9$ mm and $\beta = 60^\circ$. Using triaxial

tests performed on samples of Keswick Clay, Jaksa (1995) found $I_r = 67.4$. By using these values, Teh &

Houlsby's (1991) charts suggest that $r_p = 120$ mm and $z_p = 65$ mm, as shown in Figure 23.

These values of r_p and z_p are consistent with the value of θ_H of 100 mm obtained previously when

examining the horizontal CPT from the Keswick site. As a result, it is likely that this value of θ_H is a

manifestation of the CPT rather than a measure of the horizontal spatial variability of the clay itself. This

is discussed in greater detail by Jaksa (1995). Uzielli et al. (2005) demonstrated that the vertical scales

Figure 24. Distribution of boreholes in the KESWICK database.

of fluctuation for tip resistance and sleeve friction are different, further supporting the hypothesis that

the scale of fluctuation depends on the mode of measurement.

3.2 Large-scale characterization

In order to quantify the large-scale spatial variability of the Keswick Clay, Jaksa (1995) developed a

database, known as KESWICK, of geotechnical site investigation test measurements obtained from the

central business district of Adelaide, as shown in Figure 24. The KESWICK database contains 10,140

test data from 380 boreholes, involving a wide range of geotechnical test types. The test method with

the greatest amount of data relating to the Keswick Clay is the unconsolidated undrained (UU) triaxial

test. While this test method is less than ideal (Jaksa 1995), it is the only single test method contained

within KESWICK which provides sufficient data for large-scale, spatial variability analyses. θ 500

1000

1500

2000

2500

3000

3500

4000 0 500 1000 1500 2000 2500 Horizontal Distance, x (m)
 North-South East-West 0 50 100 150 200 250 300 0 500 1000
 1500 2000 2500 Horizontal Distance, x (m) North-South
 East-West (a) (b) N u m b e r o f P a i r s

S e

m

i v

a r

i o

g r

a m

,

x

(k

P a

)

Figure 25. (a) North-south and east-west experimental semivariograms for 0-3 metre data set; (b) number of data pairs contributing to the semivariogram calculation.

Unlike the CPT, which measures cone tip resistance, q_c , and sleeve friction, f_s , the triaxial test allows

direct measurement of the undrained shear strength, s_u , of the soil being tested. As a result, the large-scale,

spatial variability analyses presented in this sub-section are based on triaxial measurements of s_u .

As there were insufficient data to perform a complete 3D

model, it was decided to develop a pseudo

3D model. As a consequence, the s_u data were divided into 3 layers, depending on the depth of the s_u

measurement below the surface of the Keswick Clay. These layers included: (i) 0 to 3 meters; (ii) 3 to 6

meters; and (iii) beyond 6 meters below the surface of the Keswick Clay. Each of the measurements of

s_u were then averaged within these layers and along their relevant boreholes. In this way, a 'pseudo-3D'

data set was developed.

In the analyses that follow only geostatistics is used, as it provides a more convenient implementation

of 2D and 3D data than that provided by random field theory. The subsequent geostatistical analyses

were performed using the public-domain GEO-EAS software suite (Englund & Sparks 1991). Again,

as suggested by Fenton (1999a), no trend is removed. Jaksa (1995), however, did remove a trend from

these data in order to maintain consistency. The interested reader is encouraged to compare the results,

with and without trend removal, to examine the influence of such de-trending.

Preliminary investigations indicated that there were insufficient data within the second two layers to

enable appropriate analyses to be performed. Hence, by analyzing only the data contained within the

upper 3 meter layer the north-south and east-west experimental semivariograms (Fig. 25(a)), and the

omni-directional experimental semivariogram (Fig. 26(a)), were obtained. The north-south and east

west experimental semivariograms are direction-dependent, whereas the omni-directional experimental

semivariogram ignores the orientation of the pairs and

usually provides the smoothest semivariogram

(Englund & Sparks 1991). The number of data pairs contributing to their calculation is shown in

Figures 25(b) and 26(b).

Superimposed on Figures 25(a) and 26(a) is an appropriate spherical model with parameters: $a = 1,000$

meters; $C = 1,000 \text{ kPa}^2$; $C_0 = 1,500 \text{ kPa}^2$.

The north-south and east-west experimental semivariograms [Fig. 25(a)] appear to be somewhat

haphazard and are poorly represented by the spherical model. This is partly due to the data being

non-stationary within the domain of the study area (Fig. 24). This is treated in some detail by Jaksa (1995).

The omni-directional experimental semivariogram [Fig. 26(a)], on the other hand, is much smoother,

as expected, and is reasonably well represented by the spherical model, given the number of pairs

contributing to its calculation [Fig. 26(a)]. This model, therefore, suggests a large-scale horizontal range

of influence, a , equal to 1,000 m. The question arises, then, as to the reason for the discrepancy between

the values of a obtained from the small and the large-scale analyses.

It is widely accepted that the scale of spatial variability modeling may vary greatly, depending upon

the type of problem considered (Vanmarcke 1978, Journel & Huijbregts 1978), as shown in Figure 27.

As a consequence of this, Vanmarcke (1978) suggested that geotechnical properties may exhibit two, 0 500 1000 1500 2000 2500 3000 3500 4000 0 500 1000 1500 2000 2500 Horizontal Distance, x (m) 0 100 200 300 400 500 600 700 800 900 0 500 1000 1500 2000 2500 Horizontal Distance, x (m) (a) (b) Semivariogram, $\gamma(x)$ (kPa) Number of Pairs

Figure 26. (a) Omni-directional experimental semivariogram for 0-3 metre data set; (b) number of data pairs

contributing to the semivariogram calculation.

Figure 27. Scales of spatial variability modeling in geotechnical engineering: (a) sizes of soil particles; (b) sizes of

laboratory specimens; (c) vertical sampling distances; (d) lateral distances between borings; (e) horizontal intervals

measured along the centerline of long linear facilities. (After Vanmarcke 1978.)

or more, superimposed scales of fluctuation, depending on the modeling scale. From a geostatistical

perspective, Journel & Huijbregts (1978) also recognized superimposed variability, known as nested

structures. For example, the semivariogram function, $\gamma(h)$, may be expressed as the linear combination of

several separate semivariograms: It is possible to combine the two horizontal semivariogram models, from the South Parklands site

and the KESWICK database, into a single nested model. (The model from the horizontal CPT from the

Keswick site is ignored as a result of the reasons discussed above.) However, before doing so, it is

necessary to convert the South Parklands site model, which is based on q_c data, to one based on undrained

shear strength, s_u , data. In order to achieve this, the following standard CPT expression is adopted, which

is based on the bearing capacity of a deep, circular foundation (Ladd et al. 1977, Schmertmann 1978):

where σ_{v0} is the total in situ vertical overburden stress, and N_k is an assumed constant known as

the cone factor. For Keswick Clay, Jaksa (1995) found the average N_k to be equal to 17.2. Using 0 0 2 4 6 8 10 12 14 200 400 600 800 1000 1200 Horizontal Distance, x (m) 3.5 m below ground 4.5 m below ground 4.0 m below ground 5.0 m

below ground Semivariogram, γ_x (kPa²)

Figure 28. Experimental semivariograms of the horizontal su data for transect CD1 to CD50. 0 0 10 50 1000 500 1000 1500 2000 2500 γ_2 Horizontal Distance, x (m) KESWICK Data Base South Parklands Site γ_1 Semivariogram, γ_x (kPa²)

Figure 29. The two independent semivariogram models that describe the spatial variability of the undrained shear

strength of Keswick Clay. (Note: x-axis is not drawn to scale).

this relationship, Jaksa (1995) found that only C_0 and C are dependent on N_k , not a . This is con

firmed by Figure 28, which shows the experimental semivariograms of the data shown previously in

Figure 15, transformed to undrained shear strengths using Equation (28). When one compares the

undrained shear strength experimental semivariograms (Fig. 28) with those based on q_c (Fig. 17), it is

clear that only the vertical scale of the semivariograms is influenced by the transformation and, hence, a is

unaffected.

The model semivariograms from Figures 26(a) and 28 and Equations (29) and (30) are combined to

form the plot shown in Figure 29, which enables the single, nested model in Figure 30 and Equation (31)

to be formulated. 0 500 1000 1500 2000 2500 10 1000 Horizontal Distance, x (m) Semivariogram, γ_x (kPa²)

Figure 30. Nested semivariogram model that describes the spatial variability of the undrained shear strength of

Keswick Clay. (Note: x-axis is not drawn to scale.) Large-scale: Small-scale:

4 APPLICATION OF SPATIAL VARIABILITY CHARACTERIZATION TO GEOTECHNICAL ENGINEERING

The spatial variability characteristics evaluated in the previous section can be used in a variety of proba

bilistic geotechnical engineering applications. For example, Jaksa et al. (1997a) implemented the nested

semivariogram model [Fig. 30 and Eq. (31)] to predict the undrained shear strength within the Ade

laide City area (Fig. 24) at unsampled locations. Using the geostatistical technique known as kriging

(Journel & Huilbregts 1978), Jaksa et al. (1997a) found excellent agreement between the measured

values and kriged estimates. An additional benefit of geostatistics and kriging is that the error associated

with the kriged estimate can be readily determined. The interested reader is referred to Jaksa (1995) for

further treatment of kriging and its application to the KESWICK database. Another considerable benefit of random field theory and geostatistics is their ability to simulate

data that conform to the desired spatial statistics (i.e. μ , σ and θ). Fenton and Griffiths have applied

random field theory to the simulation of two and three-dimensional soil profiles in order to assess the

reliability of geotechnical systems. Using local average subdivision (Fenton & Vanmarcke 1990), which

generates accurate, robust and relatively fast random fields (Fenton 1994), in conjunction with finite

Figure 31. Typical deformed mesh of a single, rough footing on a spatially random soil. (After Griffiths et al.

2002b.)

Figure 32. Typical random field realizations and deformed mesh at slope failure for two different spatial correlation

lengths (a) $C = 0.5$; (b) $C = 2$. (After Griffiths & Fenton 2004.)

element analysis, Fenton and Griffiths adopted a random finite element method (RFEM) to assess the

reliability of: shallow foundations (Paice et al. 1996, Griffiths et al. 2002b, Fenton & Griffiths 2002, 2003, 2005); retaining walls (Fenton et al. 2005); slope stability (Griffiths & Fenton 2004); earth dam seepage (Griffiths & Fenton 1998); and underground pillars (Griffiths et al. 2002a).

As an example of the benefit of probabilistic analysis compared to its traditional deterministic counter

part, Griffiths et al. (2002b) performed a probabilistic study on the bearing capacity of a rough rigid strip

footing on a soil with randomly varying shear strength (Fig. 31). Random field theory was combined with

a conventional non-linear finite element algorithm, in conjunction with a Monte Carlo method. Griffiths

et al. (2002b) found the mean bearing capacity of a footing on a soil with spatially varying shear strength is

always lower than the deterministic bearing capacity based on the mean value. This is an important conclu

sion which results from the linking up of weak elements beneath the footing, and demonstrates that weak,

rather than strong, elements tend to dominate the bearing capacity of a footing on spatially random soil.

In another example, Griffiths & Fenton (2004) examined the stability of slopes from a probabilistic

viewpoint, again using the RFEM. One of the major limitations of deterministic, and in fact many

probabilistic, analyses is that they are often based on the classical Bishop slip circle method (Bishop

1955). The RFEM, however, models the spatially variable nature of soil deposits and facilitates the

identification of the weakest path within the soil mass, whose shape, as shown in Figure 32, is generally

more complex than the traditional slip circle. By examining

the influence of various normalized scales

of fluctuation, $C (= \sigma \ln C / H)$, where C is the soil's shear strength and H is the height of the slope),

Griffiths & Fenton (2004) found that simplified probabilistic analysis, in which spatial variability is

ignored by assuming perfect correlation, can lead to unconservative estimates of the probability of

failure. This effect is most pronounced at relatively low factors of safety or when the coefficient of

variation of the soil strength is relatively high.

Perhaps one of the most significant benefits of the application of probabilistic methods to geotech

nical engineering problems, and the associated spatial characterization of soil, is its contribution to

reliability-based design (RBD), otherwise known as load and resistance factor design (LRFD). Tradition

ally, geotechnical engineering design has been based on the working stress approach, which makes use of

factors of safety. Typically, factors of safety between 2 and 3 are considered to be adequate in the majority

of geotechnical engineering design situations. This does not, however, translate to a low or acceptable

level of risk. The enumeration of risk, on the other hand, is made possible by probabilistic methods

and, in particular, reliability analysis. Moreover, random field modeling of soil variability provides a

consistent framework for incorporating such variability in reliability-based design (Uzielli et al. 2005). As examples of this, Phoon and his co-workers presented reliability-based frameworks for the design

of drilled tension piers (Phoon et al. 2000) and shallow foundations (Phoon et al. 2003a) for transmission

line structures. The authors stated that the main advantage of RBD is that a known level of reliability can

be achieved consistently over a wide range of design scenarios and it accounts for parametric uncertainties

in a rational manner. Finally, Jaksa et al. (2003) developed a framework, based on Monte Carlo analysis, for quantifying

the reliability of site investigations as a function of soil variability, geology, and foundation and test type.

Briefly, the first step of the methodology involves simulating a 3D soil profile beneath a site, which

conforms to the specified spatial variability parameters and geology. A virtual site investigation is then

performed on the 3D simulated soil profile. Using methods commonly adopted in practice, a foundation

is then designed based on the applied loads and layout dictated by the structure and the soil properties

obtained from the virtual site investigation. The designed pile foundation is subsequently placed on the entire 3D soil profile and, because it has

been simulated, its properties are known completely and without error. Its actual performance is then

determined and compared with the original design criteria. In addition, the complete soil profile is again

used to determine the optimal pile foundation system. That is, the system that would have been designed if

the soil profile was known completely, which is never the case in reality, and is only possible because the

sites are simulated. The two foundation systems are compared with each other, that is, the one obtained

from the site investigation and the optimal one from complete knowledge of the soil profile. If the foun

dation meets the design criteria, the site investigation is deemed to have appropriately characterized the

ground. On the other hand, if the piles meet, but exceed the design criteria (beyond an applied tolerance),

the foundation is larger than it needed to be, and the construction cost differential is determined. Finally, if

the piles do not meet the criteria, the site investigation has failed to characterize the ground adequately and

costs associated with structural failure are determined. By repeating this process many hundreds of times,

within a Monte Carlo framework, the reliability of various site investigation strategies can be determined. Goldsworthy et al. (2005) and Jaksa et al. (2005) subsequently implemented the framework by exam

ining pad footing design with respect to settlement. It was observed that, in order to characterize

appropriately the sub-surface profile, a greater extent of site investigation is needed as the scale of fluc

tuation increases. This is somewhat contrary to what one might expect but is explained by the 'averaging'

effect associated with footing settlement, which results in greater footing size variability as the scale of

fluctuation increases. The strength of the framework, however, is that it enables geotechnical engineers

and clients to compare the relative merit of adopting one site investigation scheme over another, with

respect to reliability and cost.

5 CONCLUSIONS

This paper has presented the techniques commonly used to characterize the natural or spatial variability

of soil and rock and has focused on random field theory and geostatistics. It was observed that both

methods are intrinsically similar, although the semivariogram is an unbiased estimator and, unlike the

autocorrelation function, it does not depend on the mean. In order to illustrate the implementation of these methods, a case study has been presented, based

on the geotechnical properties of a stiff, over-consolidated clay from Adelaide, South Australia. It was

observed that various scales of fluctuation are present, depending on the scale of observation. It was noted that, by quantifying the mean, variance and scale of fluctuation, or range of influence, it

is possible to simulate sub-surface profiles whose spatial variability is consistent with natural deposits.

Such simulated soil profiles can be used to assess the reliability of many geotechnical systems such as

the bearing capacity and settlement of foundations, slope stability, site investigations, seepage through

earth dams and retaining walls. In addition, statistical soil characterization assists in the development of

probabilistic design methods which account for uncertainties in a robust and consistent manner.

ACKNOWLEDGEMENTS

The author is grateful for the assistance of many individuals and companies who have given generously

of their time and resources to assist with the data collection and analyses contained within this paper.

In particular, Drs. W.S. Kaggwa and P.I. Brooker, provided valuable guidance and support during and

subsequent to the author's Ph.D. studies, where the data and much of the analyses result from. In addition,

the author acknowledges his former graduate student, Jason Goldsworthy, for his tireless work in relation

to the implementation of the methodology used to examine site investigation reliability, as outlined in § 4.

The author is particularly indebted to the late Mr. Tadeusz Sawosko who spent many weeks and long,

tiring days in the field performing the more than 220 cone penetration tests. The considerable expertise

of the technical staff of the School of Civil and Environmental Engineering at the University of Adelaide,

is also gratefully acknowledged, especially Messrs. Bruce Lucas, Stan Woithe, Werner Eidam, Colin

Haese, Robert Kelman, Laurie Collins and Ms Deborah Hammond.

The field study was carried out with the generous support of the South Australian Department of

Transport and the Adelaide City Council. In particular, the author acknowledges Messrs. Roman Washyn,

Richard Herraman, Ian Forrester and Mark Underhill for their support. The horizontal cone penetration

test, carried out at the Keswick site, was enabled by the generosity and assistance of Australian National,

and in particular, Mr. Peter Gaskill. Former final year civil engineering students, Dirk van Holst Pellekaan

and Julian Cathro, carried out much of this testing under the author's supervision.

The KESWICK database was compiled as a result of the considerable generosity of the following com

panies: ACER Wargon Chapman (SA), Coffey Geosciences, Connell Wagner (SA), Golder Associates,

Kinhill Engineers, Koukourou and Partners, Rust PPK Consultants and SACON.

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Uncertainty-based characterisation of Troll marine clay

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ABSTRACT: The paper aims at providing a case-study of uncertainty-based characterisation of

selected engineering parameters in Troll marine clay. Second-moment uncertainty characterisation for

data from laboratory and in-situ tests is performed. First-Order Second-Moment (FOSM) approximation

is applied for probabilistic second-moment characterisation

of derived parameters. Spatial variability

of cone penetration testing results is investigated. A practical example regarding the characterisation of

uncertainty of undrained strength calculated from cone tip resistance is presented.

For each of the above, the sources of uncertainty are examined critically and discussed. Criteria to

evaluate the validity of results are provided. Emphasis is placed upon: (a) importance of referring to

geotechnical expertise in defining uncertainty models; (b) effort to present data in a synthetic, simple

yet rigorous form to enable seamless implementation in geotechnical design; and (c) necessity to report

explicitly, along with the results of uncertainty-based characterisation, testing methods and engineering

models used in the characterisation procedure.

1 INTRODUCTION

Hight & Leroueil (2003) effectively captured the complexity which is brought to the characterisation

of geomaterials by the presence of uncertainties, and the necessity to address such uncertainty in a

structured manner: “[...] if our geomaterials were uniform and we could run a perfect in-situ test or take

a perfect sample and run a perfect laboratory test, we would not need to consider all the factors that

might have influenced the measured soil properties. However, our soils are not uniform; we cannot run

a perfect in-situ test, take a perfect sample or run a perfect laboratory test. We must consider, therefore,

in our interpretation of the measured properties: the variability of the soils we are dealing with; how to

establish this variability; the effects of in-situ testing and the sampling of soils; and the shortcomings of

our laboratory testing.”

The importance of quantifying uncertainty in geotechnical practice is manifold. In the light of cur

rent trends in building code revision, which are increasingly relying on uncertainty-based approaches,

the two most beneficial outputs of uncertainty-based characterisation of geomaterials are possibly: (a) a

rational insight into the variability of such geomaterials; and (b) the estimation of parameters in a form

which is suitable for uncertainty-based geotechnical design. A broader overview of the implications of

uncertainty analysis in geotechnical engineering is provided in an overview paper by Uzielli et al. (2006).

It has long been recognised that the main sources of geotechnical uncertainty are aleatory uncertainty

and epistemic uncertainty (e.g. Baecher 1983). Aleatory uncertainty is inherent to physical media. Epi

stemic uncertainty (uncertainty due to imperfect knowledge) is composed of measurement uncertainty,

statistical estimation uncertainty and transformation uncertainty. Statistical estimation error results from

possible bias in sample statistics due to too few samples.

Measurement uncertainty derives from equipment, operator/procedural and random measurement

effects. Transformation uncertainty is due to the approximations and simplifications inherent in empir

ical, semi-empirical, experimental or theoretical models used to relate measured quantities to parameters

derived for engineering purposes. Epistemic uncertainty can be reduced by increasing the numerosity of

data and improved testing and analysis procedures, whereas aleatory uncertainty may remain unchanged,

or even increase, with additional data. The above sources of uncertainty can be reasonably assumed to be

uncorrelated; hence, correlation need not be addressed in total uncertainty models. A detailed overview

of the main types and sources of geotechnical uncertainty are described in Uzielli et al. (2006).

Table 1. Stratigraphic boundaries (Lunne et al. 2006).

Depth interval (m)	Unit	Soil description
0-16.5	I	clay, very soft to firm, high plasticity, slightly overconsolidated
16.5-44	IIA	clay, sandy to very stiff, medium plasticity, slightly overconsolidated, many gravels and occasional stones
44-64	IIB	clay, as unit IIA, but very stiff to hard, and slightly more overconsolidated
64-74	IIC	clay, as units IIA and IIB but less overconsolidated
74-101	IIIA	clay, sandy, very hard, highly overconsolidated, layers of fine sand and silt, many gravels and stones, low plasticity
101-135	IIIB	clay, very hard, overconsolidated, many gravels and stones, high plasticity
135-201	IV	clay, very hard, overconsolidated, a few gravels, no stones or boulders, high plasticity
201-219	V	clay, tertiary, very hard, overconsolidated, homogeneous, very high plasticity As the result of a conspicuous body of specific geotechnical research and of the interaction with

statisticians and mathematicians, uncertainty-based characterisation of engineering properties of soils

may be undertaken using a variety of approaches, a number of which are discussed by Uzielli et al.

(2006). The present paper aims at serving as a structured case-study focusing on Troll clay. To minimize

superposition of contents with the cited overview paper, the adopted methodologies are presented in an

applicative perspective rather than in a formal, comprehensive version. The emphasis of the present paper is on uncertainty-based characterisation of selected properties;

hence, it provides only a partial insight on the characterisation of Troll clay. A more complete approach,

addressing, for example, the depositional environment, post-depositional processes, stress history, com

position and structure of Troll clay, as well as complete data sets regarding additional parameters for all

soil units, is provided in Lunne et al. (2006). The paper is composed of 8 sections. Following the present introduction, Section 2 provides a general

overview of the Troll site as well as a brief explanation of the data used in the paper. Section 3 addresses

second-moment uncertainty characterisation for data measured by laboratory or in-situ tests. Section 4

illustrates the application of First-Order Second-Moment (FOSM) approximation for the probabilistic

second-moment characterisation of parameters derived from measured data; in Section 5 the spatial vari

ability of net cone resistance is investigated. Section 6 addresses the characterisation of the uncertainty of

undrained shear strength as calculated from cone tip resistance, while Section 7 illustrates an example of

Bayesian updating for the estimation of undrained shear strength using the results of laboratory (triaxial

compression) and in-situ (piezocone) testing. A general assessment of results is presented in Section 8. In practice, the presentation of results for each phase should be as concise as possible while ensuring

effective comprehension and utilisation. However, as will be seen, it is paramount that background

information regarding, for instance, the testing methods and engineering models used be provided to

ensure a reliable and conceptually correct uncertainty-based characterisation.

2 TROLL CLAY: GENERAL OVERVIEW AND DATA SOURCES

Troll is the largest gas field offshore Norway with proven reserves of more than 1300 billion standard m³

of gas. Production with export to the European continent started in 1996 and is expected to continue for

several decades. The Troll field lies in the Norwegian Trench, with water depths in range 300–330 m. The

distance to the Norwegian coast is about 65 km. Since the first soil investigation in 1983, a large number

of soil investigations have been carried out, especially for the gravity-base structure which was installed

in 1995. The clayey soils in the upper 74 m are lightly to moderately overconsolidated due to ageing,

and are very uniform across the Troll area. Below about 74 m the soils are more highly overconsolidated

due to ice loads. More details about site investigations and soil conditions are outlined in the companion

paper by Lunne et al. (2006), from where the following summary of soil stratigraphy is taken. Only units I and IIA are addressed in the present paper. These will be hereinafter referred to as UN1

(0–16.5 m) and UN2 (16.5–44 m). The separation between the soil units defined above should not be

intended as exact. The existence of a transition zone is in fact suggested by many of the data presented

hereinafter. As will be shown, the assessment of the influence of the transition zone in the context of

uncertainty characterisation requires qualitative reasoning as well as statistical analysis. Table 2. Output parameters of preliminary second-moment analysis. Symbol Description μ s Sample mean σ s Sample standard deviation COV s Sample

coefficient of variation τ_{ken} , Kendall's tau statistic for non-detrended data Stat.Ind. Assessment of statistical independence: yes (1) or no (0) based on Kendall's tau test

The laboratory data used in the present paper derive from extensive testing programs performed in

1984, 1987, 1988 and 1989. As the data were no longer available in digital format, digitisation was

performed on hard-copy plots from NGI (1989). The digitisation process, however carefully performed,

is affected by imprecision, resulting in additional uncertainty in measured values; such uncertainty can be

described as operator-related uncertainty, and is classified as measurement uncertainty. The digitisation

related uncertainty is accounted for in the following section. The piezocone (CPTU) data used are derived

from 5 soundings performed in the "Troll A" area in late 2005.

3 SECOND-MOMENT UNCERTAINTY CHARACTERISATION OF MEASURED DATA

Second-moment uncertainty characterisation of measured data, as performed herein, consists essentially

of the following steps: (a) visual inspection of data by measurement category and soil unit; (b) preliminary

second-moment data analysis; (c) identification of most suitable data mode from results of prelimi

nary second-moment analysis; (d) identification of a deterministic trend function and decomposition;

(e) identification of a suitable uncertainty model; and (f) quantification of the sources of uncertainty in

the second-moment sense, i.e. using a central (e.g. mean) value and a parameter describing the dispersion

of values around the mean (e.g. variance, standard deviation or coefficient of variation).

Visual inspection should always precede numerical treatment

of data. Its importance should never be downplayed, as it allows to identify general characters of the data set, possible evident outliers and clusters, and to obtain a general “feel” for the data set which can provide useful criteria for rational planning of statistical investigations.

Preliminary second-moment data analysis serves as quantitative support to visual inspection. It provides a useful tool to verify whether, for instance, the removal of outliers results in significant and meaningful variations in descriptive statistics. It consists of the calculation of the following statistics which may be useful in the identification of the most suitable data mode for second-moment uncertainty characterisation:

In which, given a sample of n data points $\psi_1, \psi_2, \dots, \psi_n$, the sample mean is

where n is sample numerosity (number of data points), while the sample standard deviation (taken here

as the square root of the unbiased variance) is

and the sample coefficient of variation is

The remaining statistics refer to Kendall’s tau test, a non-parametric statistical test. While Kendall’s test is

addressed in greater detail in Uzielli et al. (2006), it is sufficient to say here that the test outputs a statistic

(a) (b) (c) $X_1 \times X_1 \times X_2 \times X_2 \times X_1$

Figure 1. Intuitive representation of: (a) statistically dependent data with strong direct relationship [$\tau_{\text{ken}} \approx +1$];

(b) statistically dependent data with strong inverse relationship [$\tau_{\text{ken}} \approx -1$]; and (c) statistically independent data

[$\tau_{\text{ken}} \approx 0$].

(τ_{ken}) which can be used, along with sample numerosity, to assess whether the data points constituting

a sample are statistically independent, i.e. whether, in extremely intuitive terms, data points show (or

do not show) a trend along a given direction, as shown in Figure 1. The test statistic τ_{ken} ranges from

-1 (indicating complete inverse statistical independence) to +1 (indicating complete direct statistical

independence). In absolute value, it measures the strength of a monotonic (i.e. always increasing or

always decreasing) trend; it is therefore more general than the linear correlation coefficient, as the latter

provides only a measure of linearity. Statistical independence is extremely important in the context of

uncertainty analysis, as it implies weak stationarity, which is a necessary condition for the application

of numerous uncertainty estimation techniques as described in Uzielli et al. (2006). Following visual inspection of data for each parameter, one of the following modes is selected on the

basis of preliminary visual observation and evaluation of outputs of second-moment statistical analyses:

(a) DM1: no data are excluded from statistical analysis; and (b) DM2: one or more data values from

the transition zone are excluded from statistical analysis. Data exclusion is motivated by the fact that

even though specific values of depth of stratigraphic interfaces are provided, some gradual transition

in soil properties is to be expected at depths adjacent to layer boundaries. Such values would not be

representative of the adjacent layers; the inclusion of outliers may influence statistical analysis negatively

and to a significant extent. As the techniques used to model aleatory uncertainty require a zero-mean, weakly stationary data

set, data decomposition must be performed; this consists in the identification of a deterministic trend

function (e.g. by regression or simply as the mean of the data set), the subtraction of such trend from the

data set and the assessment of weak stationarity of the residuals of decomposition. As stated previously,

total uncertainty is derived from a number of possible uncertainty sources. An uncertainty model is

required to define how the different uncertainty components are merged to obtain a numerical estimate

of total uncertainty. The identification of a suitable uncertainty model should therefore depend from

the nature of the data (e.g. measured parameter vs. derived parameter). Here, additive models for total

uncertainty are employed. Lastly, the magnitude of the components of the uncertainty model and total

uncertainty are evaluated, examined critically and discussed. In the following, the above protocol is applied to the following properties, obtained from laboratory or

in-situ testing: (a) submerged unit weight; (b) Atterberg limits; (c) undrained shear strength from triaxial

compression and direct simple shear testing. The same approach can be used for any other property for

which data are available.

3.1 Submerged unit weight

The data set, shown in Figure 2, comprises 109 calculated values and 46 values measured directly in the

laboratory. Values were calculated assuming complete saturation and using the following relation:

Figure 2. Submerged unit weight vs. depth - all data

(DM1). Figure 3. Submerged unit weight - data set after exclusion of outliers near stratigraphic interface between UN1 and UN2 (DM2).

in which the (saturated) bulk unit weight [in kN/m^3] is calculated from the water content w , the unit

weight of solid particles γ_s [in kN/m^3] and the unit weight of water, $\gamma_w = 10.1 \text{ kN/m}^3$:

Visual inspection of Figure 2 suggests that values pertaining to UN1 are significantly distinct from those

in UN2. Failing to separate data by soil unit would result in grossly incorrect results. If data are inspected

by category, it is observed that measured values tend to yield lower results than calculated values; hence,

analysis by soil unit and category is warranted.

It is visible that there is a transition in submerged unit weight values from UN1 to UN2 at the

stratigraphic interface at 16.5 m; this is especially true for calculated values. These observations suggest

that the exclusion of data points near the stratigraphic boundary may allow for a more satisfactory

second-moment characterisation of submerged unit weight. A distance of 1.0 m above and below the

reference depths is selected; hence, data between 15.5 m and 17.5 m are excluded. The data points from

depth shallower than 5.0 m are also identified as outliers as, based on comprehensive inspection of the

available data sets from the sets, it is opined that the depth range 0.0–5.0 m could be identified as a

sub-layer of UN1.

Based on the above statements, DM2 seems to be the most appropriate data mode for statistical

second-moment uncertainty analysis. The DM2 data set is plotted in Figure 3.

To verify the above observations statistically, preliminary second-moment (SM) analyses for the two

modes are performed. Results for DM1 and DM2 are tabulated in Table 3 and Table 4, respectively. The

results of preliminary analysis for submerged unit weight are presented in detail; similar preliminary

analyses will be performed for the other parameters, but will not be presented in such detail. Table 3. Submerged unit weight - results of preliminary analysis on complete data set (DM1). Calculated Measured UN1 UN2 UN1 UN2 n 47 62 19 27 m (kN/m³) 7.1 10.4 6.8 10.3 s (kN/m³) 0.4 0.5 0.6 0.5 COV s 0.06 0.05 0.09 0.04 COV SE 0.01 0.01 0.02 0.01 τ ken,0 0.15 0.76 -0.01 0.56 Stat.Ind. 1 0 1 0 Table 4. Submerged unit weight - results of preliminary analysis on reduced data set (DM2). Calculated Measured UN1 UN2 UN1 UN2 n 38 52 17 25 m (kN/m³) 7.1 10.5 6.7 10.4 s (kN/m³) 0.3 0.3 0.3 0.4 COV s 0.04 0.03 0.05 0.04 COV SE 0.01 0.00 0.01 0.01 τ ken,0 -0.15 0.72 -0.12 0.50 Stat.Ind. 1 0 1 0

Table 5. Submerged unit weight - results of preliminary

analysis (DM1, data analyzed by category only). Calculated Measured

n 109 46

m (kN/m³) 9.0 8.9

s (kN/m³) 1.7 1.8

COV s 0.19 0.21

COV SE 0.02 0.03

τ ken,0 0.77 0.69

Stat.Ind. 0 0 Table 6. Submerged unit weight - results of preliminary analysis (DM1, data analyzed by soil unit only). UN1 UN2 n 66 89 m (kN/m³) 7.0 10.4 s (kN/m³) 0.5 0.5 COV s 0.07 0.05 COV SE 0.01 0.01 τ ken,0 0.19 0.65 Stat.Ind. 0 0 From Table 4, it follows that the mean values from calculated data exceeds those from measured data.

Moreover, in both DM1 and DM2 the sample coefficient of variation in measured data is generally greater

than that of calculated data, even though the values are very small (not exceeding 0.05). Significant

differences may exist in terms of the strength of the

spatial trend in data: in DM1, for instance, Kendall's

τ for UN1 is 0.15 for calculated data and -0.01 for measured data (see Table 3). Hence, the importance

of analyzing data by category is confirmed. In comparison with DM1, the exclusion of data outliers at stratigraphic interfaces results in a reduction

in COVs (comparing Table 3 and Table 4). The exclusion of outliers results in significant changes in

$\tau_{ken,0}$ for some units and sub-units (Table 3 and Table 4 for comparison): for UN1, for instance, there is

a strong inversion in trend (from 0.15 to -0.15) in calculated data. It has been stated previously that, whenever possible, data should be analyzed by category as well

as by soil unit. For the sake of quantitative comparison, results for the complete data set (DM1) are

shown if the data are analyzed by category only (Table 5) and by soil unit only (Table 6). It can be seen

that Table 5 is largely misleading (in comparison with Table 4, which is chosen as reference) in terms

of mean, sample COV and $\tau_{ken,0}$. Table 6 provides results which are closer to Table 4; nevertheless, the

differences between categories in terms of second-moment statistics, trend type and magnitude are not

captured. It should be noted that assessment of statistical independence cannot be made from the values

of τ_{ken} only, as critical values of such parameter depend on sample numerosity.

The following model for the total coefficient of variation, representative of total uncertainty, is

proposed:

in which COV ω is the coefficient of variation of inherent variability, representative of aleatory uncer

tainty; COV m is the coefficient of variation of

measurement error; and COV SE is the coefficient of

variation of statistical estimation uncertainty.

In the following, the components of total uncertainty in submerged unit weight are estimated. In the

preliminary analysis, both calculated and measured data were considered. In the subsequent phase of

second-moment uncertainty characterisation, only measured data are addressed. This choice is made as

the calculated values are affected by additional transformation uncertainty (which, moreover, is difficulty

to quantify) deriving from the transformation model in Eq. (4).

In the estimation of aleatory uncertainty for a property ψ , decomposition is an important preliminary

operation, consisting in the identification of a deterministic trend function and the subtraction of such

function from the data to obtain a zero-mean set of residuals:

in which s is a generic spatial direction (e.g. depth); $\psi(s)$ is a generic measured property along s ; $t \psi(s)$ is

a deterministic trend function obtained by regression or other means; and $\xi \psi(s)$ is a zero-mean, weakly

stationary set of residuals whose aleatory uncertainty can be estimated using appropriate methods. In

this case, $\psi = \gamma'$ and $s = z$ (i.e. depth). As detailed in Uzielli et al. (2006), the magnitude of aleatory

uncertainty depends on the set of residuals resulting from decomposition; hence, from the removed

trend. Therefore, it is paramount: (a) to verify whether decomposition by regression is necessary; or

whether it is sufficient to subtract the sample mean from the data; and (b) if decomposition by regression

is necessary, that the trend function be reported.

Kendall's tau test (yielding a 95% confidence estimate of statistical independence) can be used to

investigate (a). If data are found to be statistically independent prior to trend removal, no detrending

(except for the subtraction of the mean value to obtain zero-mean residuals) is necessary. As for point (b),

while no unique criterion for trend identification exists, the lowest-degree polynomial trend which

generates statistically independent residuals is identified as candidate trend function. Subsequently, it

should be verified that such trend: (1) yields plausible [e.g. non-negative] values; (2) can be related to

some physical reason [e.g. overburden stress increasing linearly with depth]; and (3) does not conflict

with geotechnical engineering knowledge.

It is seen in Table 4 that performing Kendall's tau test yields the result by which data in UN1 are

statistically independent, while data in UN2 are not; hence, no detrending by regression is necessary

for UN1 data (a constant trend corresponding to the value of the sample mean is assumed), while it is

necessary for UN2. The following linear trends are defined for UN1 and UN2, respectively:

in which γ' is in kN/m³ and z is in m. It should be noted that the trend given by Eq. (8) is constant, while

the trend given by Eq. (9) increases linearly with depth.

Performing Kendall's test on the detrended data sets yields $\tau_{ken} = -0.15$ and $\tau_{ken} = 0.04$ for UN1 and

UN2, respectively. Such values result in a positive assessment of statistical independence for both soil

units. Hence, the first-degree polynomial trends defined by Eq. (8) and Eq. (9) are accepted as reference

trends. The results of decomposition, obtained by applying Eq. (7), are shown in Figure 4b.

The coefficient of variation of inherent variability, which is representative of aleatory uncertainty (not

accounting for spatial averaging effects) is calculated for each soil units as:

in which s_{ξ} is the standard deviation of the zero-mean set of residuals ξ ; and μ_t is the mean value of

the trend function in the spatial extension (soil unit) under investigation. As the analysis is carried out at

soil unit-level, COV ξ is representative of the entire soil unit. This implies that the standard deviation of

the residuals is assumed to be constant over the soil unit. This assumption is related to the condition of Table 7.

Second-moment estimates of uncertainty components and total uncertainty for submerged unit weight. UN1 UN2 μ_t (kN/m³) 7.1 9.4 COV ξ 0.04 0.02 COV m 0.03 0.03 COV SE 0.01 0.00 COV tot 0.05 0.04 σ_{tot} v (kN/m³) 0.3 0.4

homoscedasticity (by which the scatter of points around the trend is homogeneous throughout the soil

unit), which is necessary for performing regression on the data. In the present case, applying Eq. (10) to the residuals related to the trends defined by Eq. (8) and Eq. (9)

yields COV ξ = 0.04 for UN1 and COV ξ = 0.02 for UN2; these values are lower than those obtained in

the preliminary analysis and shown in Table 4. The data set under investigation consists of measured values of submerged unit weight. Numerical

values were retrieved from hard-copy plots through digitisation. Hence, in the present case, measurement

uncertainty consists of two components: (a) measurement error deriving from imprecision and inaccuracy

in laboratory testing; and (b) imprecision in data digitisation. The following measurement uncertainty

model is assumed:

in which COV_{tst} is the coefficient of variation of testing error and COV_{dig} is the coefficient of variation

of digitisation error. A value of 0.01 is assigned to COV_{tst} (γ') for submerged unit weight based on

Phoon & Kulhawy (1999a). On the basis of calibration purposely performed between digitised data and

available tabulated data, the coefficient of variation to account for digitisation imprecision is quantified

as $COV_{dig}(\gamma') = 0.03$. Hence, from Eq. (11), $COV_m(\gamma')$ ≈ 0.03 . The statistical estimation error related to trend identification as described previously is given by

in which COV_{ξ} is the coefficient of variation of inherent variability and n is the sample numerosity. In

the present case, as shown in Table 4, $n = 17$ for UN1 and $n = 25$ for UN2; hence, rounding to the nearest

percentile value, $COV_{SE}(\gamma') \approx 0.01$ for UN1 and $COV_{SE}(\gamma') \approx 0.00$ for UN2. Transformation uncertainty is not present in this case, as source data are direct laboratory

measurements. From Eq. (6), $COV_{tot}(\gamma_{eff})$ is estimated to be 0.05 for UN1 and 0.04 for UN2. Table 7 reports second

moment estimates of uncertainty components and total uncertainty for submerged unit weight. The mean

values of such trends (μ_t) are shown as they are necessary in the calculation of the total standard deviation

for each soil unit: Figure 4 shows the results of linear regression on data from UN1 and UN2. The linear trends described

by Eq. (8) and Eq. (9) are shown, as well as ± 1 total standard deviations. It may be seen that the hypothesis

of homoscedasticity (i.e. constant standard deviation of residuals with depth) seems to be satisfactorily

verified for both soil units.

3.2 Atterberg limits

Laboratory measurements are available for plastic limit (w_P) and liquid limit (w_L); the complete set are

shown in Figure 5. In the DM1 mode, whose results are shown in Table 8, it is seen that the sample

mean of w_P and w_L in UN1 are significantly higher than in UN2; also, sample coefficients of variation

indicate a higher dispersion around the central value in UN1 than in UN2. However, the low numerosity

of data in the samples should be acknowledged.

Figure 4. Results of decomposition and second-moment analysis for submerged unit weight: (a) original data, trends

and standard deviations; (b) residuals.

Visual inspection of Figure 5 suggests that the data points measured at 16.3 m would belong to a

transition zone rather than being representative of UN1. Here, it is deemed that they should be identified

as outliers and excluded from the set. Hence, DM2 (shown in Figure 6) is selected as reference analysis

mode for w_P and w_L . The low values of COV w in Table 8 indicate that Atterberg limits are substantially

uniform in UN2.

Trends are not evident in Figure 6; moreover, $\tau_{ken,0}$ may not be a reliable discriminatory parameter in

this case as sample sizes are very low. Hence, mean values are assumed as constant trend functions for

both parameters and for both soil units.

Phoon & Kulhawy (1999a) reported coefficients of variation of testing error (COV tst) of 0.07 and

0.10 for w_L and w_P , respectively. The imprecision in digitisation is also accounted for, as has been done

for submerged unit weight. Values of COV m can be obtained

using Eq. (11), i.e. adopting the same uncertainty model defined for submerged unit weight.

Statistical estimation uncertainty can be quantified using Eq. (12).

Table 10 reports output second-moment estimates of uncertainty components and total uncertainty

for w_P and w_L . Here, too, the low number of data points, especially in UN1, should be considered.

3.3 Undrained shear strength

Laboratory data from triaxial compression (CAUC) and direct simple shear (DSS) tests are addressed.

The complete data sets are plotted in Figure 8.

No outliers at stratigraphic boundaries are evident from visual inspection of Figure 8; however, data

in the range 0.0–5.0 m are once again excluded. Hence, DM2 is judged to provide the most appropriate

analysis mode. Results of preliminary second-moment analyses on undrained shear strength measure

ments for mode DM2 are shown in Table 11. It should be noted that the high $\tau_{ken,0}$ values in Table 11

indicate strong trends in data; this is not surprising as undrained shear strength increases with effective

stress and depth for normally and slightly overconsolidated clay. The higher values of $\tau_{ken,0}$ in UN1 may Figure 5. Atterberg limits – complete data sets (DM1). Figure 6. Atterberg limits – reduced data sets (DM2).

Table 8. Atterberg limits – results of preliminary

analysis on complete data set (DM1). w_P w_L UN1 UN2 UN1 UN2

n 3 4 3 4

m (%) 20.70 14.03 53.70 33.65

s (%) 4.17 0.72 11.80 0.67

COV s 0.20 0.05 0.22 0.02

$\tau_{ken,0} = -0.50 - -0.33$

Stat.Ind. = 1.00 = 1.00 Table 9. Atterberg limits - results of preliminary analysis on reduced data set (DM2). w P w L
UN1 UN2 UN1 UN2 n 2 4 2 4 m (%) 22.9 14.0 59.9 33.7 s (%)
2.4 0.7 6.9 0.7 COV s 0.10 0.05 0.12 0.02 $\tau_{ken,0} = -0.50 - -0.33$ Stat.Ind. = 1.00 = 1.00

reflect the higher overconsolidation ratio and plasticity index of such unit, as described by Lunne et al.

(2006). In the second-moment characterisation of undrained shear strength, the total uncertainty model given

by Eq. (6) is again referred to. Decomposition must be performed in order to estimate aleatory uncertainty.

As previously stated, strong trends are evident in Figure 8, and Kendall's tau test confirms that detrending

by regression is necessary. For design purposes, a constant, minimum value is usually assigned to undrained shear strength at

shallow depths; here, the trends obtained by regression do not apply for depths smaller than 5.0 m as

data from shallower depths were excluded. The trend functions defined by the equations in Table 12 are

identified for CAUC data for UN1 and UN2. The weak stationarity of residuals is assessed by Kendall's tau test prior to acceptance of the equations

in Table 12. The results of decomposition are shown in Figure 9 and Figure 10 for CAUC and DSS,

respectively. For CAUC data, the condition of homoscedasticity (i.e. constant standard deviation of

residuals with depth) seems to be well met for UN2; in UN1, however, there appears to be an increase in

Figure 7. Results of decomposition and second-moment analysis for Atterberg limits: (a) original data, trends and

standard deviations; (b) residuals. Table 10. Second-moment

estimates of uncertainty components and total uncertainty for Atterberg limits. w P w L UN1 UN2 UN1 UN2 μ t (%) 22.9 14.0 59.9 33.7 COV ξ 0.10 0.05 0.12 0.02 COV m 0.10 0.10 0.08 0.08 COV SE 0.07 0.03 0.08 0.01 COV tot 0.17 0.12 0.16 0.11 σ tot (%) 3.9 1.7 9.6 3.7

the scatter of residuals with depth. In the DSS data, scatter visibly increases with depth in UN2. Hence, the

results of regression for undrained shear strength data should be viewed somewhat more cautiously than

the ones obtained for submerged unit weight.

It is seen in Figure 9 and Figure 10 that there is a discontinuity in trend values at the stratigraphic

interface between UN1 and UN2, with higher undrained shear strength in UN1. This is consistent with

the values reported by NGI (1989) and with the results of FOSM modelling of the plasticity index, which

will be addressed further on in the paper.

The aleatory uncertainty in the residuals related to the trends reported in Table 12 is estimated using

Eq. (10) for CAUC and DSS, and for each soil unit. Results are shown in Table 13.

Phoon & Kulhawy (1999a) reported literature values for the coefficient of variation of measurement

error in undrained shear strength determined by triaxial compression; such values ranged from 0.08

Figure 8. Undrained shear strength vs. depth - complete data sets for CAUC and DSS (DM1).

Table 11. Undrained shear strength - results of

preliminary analysis for reduced data set (DM2). CAUC DSS
UN1 UN2 UN1 UN2

n 22 33 23 34

m(kPa) 27.5 80.5 23.9 69.2

s(kPa) 11.0 29.0 12.1 19.4

COV s 0.40 0.36 0.51 0.28

$\tau_{ken,0}$ 0.90 0.89 0.76 0.77

St. ind. 0.00 0.00 0.00 0.00 Table 12. Trend functions for undrained shear strength by testing method and soil unit (in kPa, for $z > 2.5$ m, z in metres). Test Soil unit
Equation of trend CAUC UN1 t UN1 (s uCAUC) = $2.65z - 1.0$
UN2 t UN2 (s uCAUC) = $3.52z - 19.6$ DSS UN1 t UN1 (s uDSS) = $2.63z - 3.7$ UN2 t UN2 (s uDSS) = $2.95z - 12.2$

to 0.38. A mean value of 0.19 was suggested. For undrained shear strength measurements from direct

simple shear tests, values of COV τ_{st} indicated by Phoon & Kulhawy (1999a) ranged from 0.19 to 0.20,

with a suggested value of 0.20. Accounting for imprecision in digitisation again leads to the adoption

of the measurement uncertainty model in Eq. (11). It is worthy to note that characterising uncertainty

for the entire soil unit using a single coefficient of variation implies that higher standard deviations are

expected for higher measured values of undrained shear strength. Though the degree of approximation

inherent in this statement is evident, the variable magnitude of standard deviation is consistent with the

necessity, in triaxial testing practice, to make use of different apparatus components and measurement

devices of differing precision depending on preliminary indications of undrained shear strength provided

by past experience or index testing (Dyvik, personal communication). Statistical estimation uncertainty

is again estimated using Eq. (12). Table 13 reports second-moment estimates of uncertainty components and total uncertainty for CAUC

and DSS.

Figure 9. Undrained shear strength from CAUC testing - results of second-moment characterisation: (a) trends and

standard deviations; (b) residuals of detrending.

Figure 10. Undrained shear strength from DSS testing – results of second-moment characterisation: (a) trends and

standard deviations; (b) residuals of detrending. Table 13. Second-moment estimates of uncertainty components and total uncertainty for undrained shear strength. CAUC DSS UN1 UN2
UN1 UN2 μ t (kPa) 28.4 80.5 25.8 69.2 COV ξ 0.09 0.06 0.21 0.11 COV m 0.20 0.20 0.20 0.20 COV SE 0.02 0.01 0.04 0.02 COV tot 0.22 0.21 0.29 0.23 Table 14. Available CPTU soundings: numerosity, maximum and minimum depths, geographical coordinates and water depth. Name n z min (m) z max (m) E N z w (m) CPTU101 1501 0.00 15.00 0543149.3 6722198.2 306.7 CPTU102 1494 0.00 14.90 0543249.3 6722198.4 309.2 CPTU103 1484 0.00 14.83 0543358.0 6722299.2 307.8 CPTU104 1475 0.00 14.75 0543382.0 6722211.8 308.4 CPTU105 1383 0.00 13.85 0543273.9 6722211.1 317.8 It may be seen that the total uncertainty in UN2 is smaller than in UN1; this is due to the smaller

aleatory and statistical uncertainties. The significantly higher uncertainty in DSS (deriving primarily

from the high aleatory uncertainty in DSS measurements for UN1) should also be noted. Overall, total

uncertainty is moderately high for both tests; the effect of measurement uncertainty is significant. As may be inferred both visually, from the trend equations in Table 12 and from Table 13, the results

of second-moment characterisation of undrained depend significantly on the data set (CAUC or DSS).

In this case, as the measurement uncertainties are coincident, the difference in total uncertainty is mostly

due to aleatory uncertainty; however, in most cases, the effect of measurement uncertainty is relevant.

Moreover, aleatory uncertainty is directly related to the selected trend function. Hence, it is paramount

to report trends and reference testing method explicitly when presenting results of uncertainty-based

characterisation.

3.4 CPTU cone tip resistance and pore pressure

Results of 5 offshore CPTU investigations from the "Troll A" area (denoted by CPTU 101CPTU 105)

are available for UN1. Data were obtained using a 10 cm² tip area cone; measurement interval is 0.01 m.

Some relevant data regarding the soundings are reported in Table 14. Data sets of cone tip resistance (q_c)

and pore pressure measured behind the cone (u_2) are addressed hereinafter. It should be noted that, as shown in Table 14, water depth for the different soundings range from

306.7 m to 317.8 m. However, it is of interest to address the 5 soundings cumulatively, as this allows to:

(a) obtain average values which could be more representative than those deriving from single soundings;

(b) assess the degree of homogeneity between the measurements from different soundings. To perform such cumulative analyses, it is assumed that soil stratigraphy follows the seabed profile

with good approximation, and that profiles can consequently be offset to a common zero-depth with

only limited uncertainty; such uncertainty is addressed as described in the following. Given the above assumptions, data from the 5 soundings can be plotted as shown in Figure 11.

A map showing the locations of CPTU soundings from the 2005 site investigation is shown in Figure 12.

Qualitative visual inspection of Figure 11 suggests that:

(a) the profiles denote a considerable regularity

and smoothness; (b) despite the distances between the sounding locations (reported in Table 15), there

is considerable overlap between the q_c and u_2 profiles. These observations attest for strong homogeneity

in the horizontal direction. Spatial correlation in the horizontal direction cannot be investigated reliably

due to the limited number of soundings and the considerable spacing between the soundings themselves

(see Table 15). At each measurement depth from the common zero-depth, the sample mean and standard deviation of

the measurements from the 5 soundings (and, hence, the sample coefficient of variation) are calculated.

The scatter in the data constituting each sample can be ascribed to the inherent variability of the penetrated

Figure 11. Superimposed plots of: (a) cone tip resistance and (b) pore pressure for 5 reference soundings.

Figure 12. Map showing CPTU locations from 2005 site investigation. Table 15. Distances between CPTU soundings (km). Name CPTU101 CPTU102 CPTU103 CPTU104 CPTU105 CPTU101
0.000 0.712 3.473 1.859 1.079 CPTU102 * 0.000 3.218 1.175
0.489 CPTU103 * * 0.000 2.715 2.771 CPTU104 * * * 0.000
0.808 CPTU105 * * * * 0.000

soil and to measurement error. Phoon & Kulhawy (1999a) suggested a value of 0.07 for the coefficient

of variation of cone tip resistance measurement error in piezocone testing. While the authors are not

aware that any corresponding estimate is available in the literature for u^2 , an investigation by NGI (2002)

suggests that a value of 0.07 may also be assumed for pore pressure measurements behind the cone.

Lastly, as the numerosity n_c of the samples is limited (at most 5) and sample standard deviations may be

significant, statistical estimation error must be accounted for. Transformation uncertainty is conceptually

not pertinent, as no derived parameter is sought in this phase of the procedure. The total coefficient of variation can be obtained for q_c and u^2 at each measurement depth using the

following model:

in which COV_{ξ} , D is the coefficient of variation of aleatory uncertainty; COV_m is the coefficient of

variation of cone tip resistance measurement error; $COV_{SE,D}$ is the coefficient of variation of statistical

uncertainty and COV_{off} is an additional uncertainty term which accounts for the artificial offsetting of

CPTU soundings performed at heterogeneous water depths. An objective quantification of the uncertainty

deriving from the zero-offsetting procedure would be most difficult; hence, based on Harr's (1987) "rule

of thumb" guidelines, a value of 0.20 (corresponding to "medium" dispersion) is assigned subjectively

to COV_{off} . The use of the "D" subscript in Eq. (14) emphasizes the fact that while aleatory uncertainty and

statistical estimation uncertainty are depth-dependent, the coefficient of variation of measurement error

and the offset-related uncertainty term are assumed to remain constant. Performing characterisation at depth level allows to appreciate the variability in total uncertainty

with depth. Total uncertainty is not constant with depth because: (a) inherent variability is generally

variable with depth; (b) a different number of measurements may be available at greater depths (some

soundings are deeper than others). The above sources of uncertainties contribute to the "depth-level"

total uncertainty. Figure 13 and Figure 14 show (for q_c and u_2 , respectively): (a) superimposed data sets from the

5 available CPTU soundings, with the average value calculated at each measurement depth; (b) the

coefficients of variation representative of the components of total uncertainty, as well as the coefficient of

variation of total uncertainty; and (c) the contours of minimum and maximum values, with superimposed

mean and the standard deviation

calculated at each depth.

4 FOSM APPROXIMATION OF DERIVED PARAMETERS

The object of the present section is the application of First-Order Second-Moment (FOSM) approxima

tion (e.g. Melchers 1999) for the probabilistic second-moment characterisation of parameters derived

from measured data - namely: total overburden stress, plasticity index, net cone resistance. The basis of

the FOSM approach lies in the statement that satisfactory estimates of the parameters of the probability

distribution of a random variate may be given by first-order approximations of Taylor series expansions

of second-moment parameters (mean and variance) calculated from samples. FOSM approximation is a useful and efficient means for estimating second-moment characteristics

of random variables which are functions of - or derived from - other random variables when it is not

possible, pertinent or of interest to obtain information regarding the distributions of the input and/or and

output variates.

Figure 13. Depth-level second-moment characterisation of cone tip resistance for CPTU data sets: (a) superimposed

plots from 5 soundings; (b) depth-level coefficients of variation; (c) depth-level mean and standard deviation.

Figure 14. Depth-level second-moment characterisation of measured pore pressure for CPTU data sets: (a) super

imposed plots from 5 soundings; (b) depth-level coefficients of variation; (c) depth-level mean and standard

deviation. In the following, the results of second-moment characterisation obtained previously will be used to

obtain second-moment estimates of total overburden stress, plasticity index and net cone resistance.

4.1 Total overburden stress

Total overburden stress is given by

in which γ is the (saturated) bulk unit weight, γ' is the submerged unit weight, $\gamma_w = 10.1 \text{ kN/m}^3$ is the

unit weight of water and z is the depth below seabed. Total overburden stress at each depth can be described in the second-moment sense based on the results

of second-moment characterisation of submerged unit weight. The mean value and standard deviation

are given by Eq. (17) and Eq. (18), respectively:

in which

is the mean value of saturated unit weight, $\mu_D(\gamma')$ is the mean value of submerged unit weight and

$\text{COV}_{\text{tot}}(p_0)$ is the total coefficient of variation of p_0 . The notation in Eq. (17), Eq. (18) and Eq. (19)

reflects the fact that total overburden stress is addressed at depth-level. This follows from the fact that

$\mu_D(\gamma')$ (corresponding to the values of the trends defined by Eq. (8) and Eq. (9)) is also depth-dependent. In Eq. (16), γ' is the only random variable; hence, it can be shown that the standard deviation of γ is

equal to that of γ' . However, it is important to note that the results of second-moment characterisation of

submerged unit weight were defined, for UN1, only in the depth range 5.0–16.5 m. In order to calculate

total overburden stress, it is necessary to make the hypothesis that such results can be extended to the

depth range 0.0–5.0 m. To account for this additional uncertainty, a coefficient of variation COV_{ovs} ,

accounting for the uncertainty in the submerged unit weight of the overlying soil, is defined; a value of

0.05 is assigned subjectively on the basis of the belief that the unknown value of γ' in the depth range

0.0–5.0 m is not significantly distinct from that of the

5.0–15.0 m depth range. The total coefficient of variation of p_0 in UN1 can then be calculated using the following proposed

additive model:

The total coefficient of variation of γ' was estimated at 0.05 for UN1; hence, from Eq. (20),

$COV_{tot}(p_0) \approx 0.06$. Even though the above coefficient of variation is assumed to be constant in UN1,

the standard deviation of total overburden stress is depth-dependent: the uncertainty at greater depths

is larger than at shallow depths because of the increasing number of uncertain values of unit weight of

overlying soil strata which contribute to overburden stress increase. Figure 15 shows the mean $\pm$ standard

deviation curves for: (a) saturated unit weight; and (b) total overburden stress. It should be noted that

spatial averaging effects are neglected in the above formulation, as it is difficult to investigate the spatial

correlation structure of unit weight; this leads to a (conservative) overestimation of $COV_{tot}(p_0)$.

4.2 Plasticity index

The plasticity index is given by

As w_L and w_P have been modelled as random variates, it is possible to characterize the uncertainty

in plasticity index in the second-moment sense using FOSM approximation. The expected value and

variance of a linear combination Y of random variates $X_1, \dots, X_n$ are

Figure 15. Second-moment characterisation of saturated

unit weight and total overburden stress. Figure 16. Soil unit-level second-moment characterisation of plasticity index.

in which k_i are scalar coefficients, are given

respectively by (e.g. Melchers 1999):

where σ_{X_i} is the standard deviation of X_i and ρ_{ij} is the linear correlation coefficient between X_i and X_j ,

with $\rho_{ii} = 1.00$. Equation (21) is obtained from Eq. (22) by setting $X_1 = w_L$, $X_2 = w_P$, $k_1 = 1$ and $k_2 = -1$.

Using Eq. (23) and Eq. (24), the expected value and variance of the plasticity index are obtained:

It should be remarked that there are only two data sets in UN1. Hence, it is not conceptually pertinent

to address correlation between such values. A conservative estimate of variance is obtained for UN1 by

setting $\rho(w_L, w_P) = 0$. On the basis of the values of second-moment estimates obtained previously for w_L

and w_P and shown in Table 10, and obtaining $\rho(w_L, w_P) = 0.22$ for UN2, the second-moment descriptive

parameters of the plasticity index are $\mu(I_P) = 37.0\%$, $\sigma(I_P) = 10.4\%$ for UN1; and $\mu(I_P) = 19.6\%$,

$\sigma(I_P) = 4.4\%$ for UN2. The soil unit-level second-moment characterisation of the plasticity index is

shown in Figure 16. Mean values of w_L , w_P and I_P are shown as continuous lines; standard deviations

for w_L and w_P are shown as dotted lines; standard deviations for I_P are shown as dashed lines.

4.3 Net cone resistance

In CPTU testing, net cone resistance (e.g. Lunne et al. 1997) is given by

in which q_c is the measured cone tip resistance; u_2 is the pore pressure measured behind the cone; p_0 is

the total vertical overburden stress; and a_c is the cone area ratio. The cone area ratio for the cone used

in the soundings performed in 2005 is $a_c = 0.75$. It has been shown how q_c , u_2 and p_0 have been modelled as random variates and their uncertainties

described in the second-moment sense. It is now possible to characterize net cone resistance as a random

variate using FOSM approximation. Here, FOSM characterisation of q_{net} is performed at depth level

rather than at soil unit level. Hence, data sets at each measurement depth can be regarded as samples of

realisations of q_c , u^2 and p_0 . In the present case, Eq. (27) is reconducted to Eq. (22) by setting the following: $X_1 = q_c$; $X_2 = u^2$;

$X_3 = p_0$; $k_1 = 1.00$; $k_2 = (1 - a_c) = 0.25$; and $k_3 = -1.00$. The mean and variance of q_{net} at each

measurement depth are obtained from Eq. (23) and Eq. (24):

In this case, FOSM analysis is performed at depth-level rather than at soil unit-level; hence, it is

conceptually acceptable to set linear correlation coefficients between distinct variates equal to zero; this

provides a conservative (higher) estimate of variance. Thus, Eq. (29) becomes:

It should be remarked that, while the total coefficients of variation of q_c and u^2 are depth-dependent,

that of total overburden stress is constant. The total coefficient of variation of q_{net} can be calculated as

follows:

Figure 17 shows: (a) the variation of the total coefficient of variation of q_{net} with depth; and (b) the

variation in $\mu_D(q_{net})$ and $\mu_D(q_{net}) \pm \sigma_D(q_{net})$ with depth.

4.4 Cone factor

As discussed in Uzielli et al. (2006), neglecting spatial averaging effects may lead to overestimation

of total uncertainty for design purposes. If the total design uncertainty in undrained shear strength is

of interest, it should be remarked that spatially averaged

design value of s_u cannot be obtained from

laboratory testing data, as the scale of fluctuation cannot be calculated reliably from such data due to the

facts that measurement depth interval is variable and excessively large. Hence, it is necessary to rely on

indirect estimates of strength derived from CPTU data. This introduces transformation error due to the

inevitable approximation of models and relations. The well-known relation between net cone resistance

and undrained shear strength is given by:

in which $s_{u\text{CPTU-CAUC}}$ is the undrained shear strength calculated using the cone factor $N_{kt\text{CAUC}}$. Such cone

factor is given by the ratio of net cone resistance to CAUC-measured undrained shear strength:

Figure 17. Depth-level second-moment representation of net cone resistance: (a) total coefficient of variation;

(b) depth-level variations of mean and mean $\pm$ standard deviation.

Here, the cone factor is characterised in the second-moment sense. From FOSM approximation, the

expected value of a ratio Y of random variates X_1 and X_2

can be estimated approximately by

while the variance is approximated by:

A point value of the cone factor can be calculated using FOSM approximation for each value of $s_{u\text{CAUC}}$

measured in the laboratory, provided that a value of q_{net} can be calculated at the same “nominal”

measurement depth. The measurement depth associated with CAUC measurements of undrained shear

strength is taken as the reference nominal depth. A total of $n_s = 17$ measurements of $s_{u\text{CAUC}}$ are available

for UN1.

A detailed analysis of the components of uncertainty in q_{net} and s_{uCAUC} is warranted. For the former

parameter, a brief excursus into cone penetration resistance is warranted. Previous research efforts on the

physical phenomena involved in cone penetration have suggested that cone tip resistance is not a point

measurement; rather, it is representative of the resistance of a volume of soil adjacent to the cone tip at

each stage of penetration. It would thus seem advisable to consider the average of q_{net} values over a spatial

extension. While a univocal criterion to quantify the dimensions of the soil volume contributing to cone

resistance at a reference depth is not available, it should be remembered that CAUC measurements are

not coming from the same locations as CPTU soundings; hence, the difference in depth measurement,

deriving from the fact that layers are most probably not parallel to seabed. A window of 1.00 m of data,

centered at the nominal depth, is deemed adequate. Hence, as measurement interval for the 5 soundings

is 0.01 m, there are $n_q = 101$ values in each of the n_s sample of q_{net} values. Besides providing a more

physically consistent representation of cone tip resistance, consideration of a spatially representative

value allows to reduce statistical estimation error in the q_{net} value used to estimate N_{ktCAUC} . As the analysis is performed at depth-level, it is again possible to set all correlation coefficients equal

to zero in Eq. (35) and Eq. (36); hence, setting $X_1 = q_{net}$ and $X_2 = s_{uCAUC}$ in Eq. (35) and Eq. (36), the

mean of the j -th calculated value of cone factor $N_{k,j}$ ($j = 1, 2, \dots, n_s$) is given by:

in which $s_{uCAUC,j}$ is the j -th CAUC-measured value of undrained shear strength; and

is the mean of the $n_q = 101$ $q_{net,i(j)}$ values in the j -th sample (i.e. centered at the nominal depth of $s_{u,j}$). The

coefficient of variation of the j -th estimated cone factor is obtained from Eq. (36), in which $p \times 1, X^2 = 0$:

in which $COV_{tot, (q_{net,j})}$ and $COV_{tot} (s_{uCAUC,j})$ are the total coefficients of variation of q_{net} and s_{uCAUC} ,

respectively calculated at each depth at which a s_{uCAUC} measurement is available. The calculation is

detailed in the following. The coefficient of variation of net cone resistance to be used in Eq. (39) is

given by

in which

is the standard deviation, calculated from the $n_q = 101$ $q_{net,i(j)}$ values ($i = 1, 2, \dots, n_q$) in the j -th data

window. Such coefficient of variation accounts for the uncertainty due to: (a) the spatial (as opposed to

punctual) nature of net cone resistance and undrained shear strength measurements; and (b) the depth

difference between the CAUC test and the average q_{net} value. The uncertainty due to the variable water

depths of the CPTU soundings, used to calculate the average net cone resistance, has been accounted

for in the total uncertainty of q_{net} itself. Undrained shear strength values are affected by measurement error and imprecision in digitisation. In

cone factor estimation, values are addressed individually; hence, no inherent variability and statistical

estimation error are present. Moreover, as undrained shear strength is measured directly, no transfor

mation uncertainty arises. As detailed in Table 13, the total coefficient of variation for each of the n_s

CAUC-measured values s_{uCAUC} , to be used in Eq. (39), is $COV_{tot} (s_{uCAUC,j}) \approx 0.20$. Equation (37) and Eq. (39) are

applied to the n s resulting pairs of s uCAUC and q net data, yielding a central

value $\mu(N \text{ ktCAUC}, j)$ and a dispersion parameter $COV(N \text{ ktCAUC}, j)$ at each of the reference measurement

depths. However, for design purposes, it would be beneficial to define $N \text{ ktCAUC}$ in the second-moment

sense at soil unit-level, so that a single central value and uncertainty parameter can be applied at all

depths inside UN1.

The central soil unit-level value is given by

i.e. by the arithmetic mean of the individual nominal values obtained at the reference measurement

depths. The sources contributing to uncertainty in the soil unit-level nominal value are: the dispersion in

the 17 central values; and (b) the uncertainty in the individual central values. The following model for

the total coefficient of variation of $N \text{ ktCAUC}$ at soil unit-level is proposed:

in which

is the coefficient of variation due to data scatter, with the sample standard deviation

calculated from the sample of 17 central values $\mu(N \text{ ktCAUC}, j)$. The coefficient of variation of statistical

estimation error of $\mu(N \text{ ktCAUC}, UN)$ is given by

To account for the total uncertainty in the individual second-moment estimates of $N \text{ kt}$, the sample mean

of the coefficients of variation is calculated:

Such parameter is representative of the uncertainty in the depth-level estimates of cone factor. The

statistical estimation error associated with the calculation of such mean value is given by

in which

is the standard deviation of the sample of n s coefficients of variation of N_{ktCAUC} . From the above uncertainty

model, the following second-moment values (applicable at all depths inside UN1) are obtained

for N_{kt} : $\mu(N_{ktCAUC,UN}) = 14.3$; $COV(N_{ktCAUC,UN}) = 0.29$, from which $\sigma(N_{ktCAUC,UN}) = 4.1$.

Figure 18 shows, at each of the 17 reference measurement depths in UN1: (a) nominal values and

total standard deviations for s_{uCAUC} ; (b) nominal values and total standard deviations for q_{net} ; and

(c) values and total standard deviations, as well as the soil unit-level (terminal) second moment parameters, for N_{ktCAUC} . It can be seen in Figure 18c that all

expected values of N_{ktCAUC} are included within one soil unit-level standard deviation $\sigma(N_{ktCAUC,UN})$ of the soil unit-level mean $\mu(N_{ktCAUC,UN})$.

Figure 18. First-order second-moment characterisation of cone factor: (a) second-moment results for

s_{uCAUC} -measured undrained shear strength; (b) second-moment results for net cone resistance; (c) depth-level FOSM

estimates and second-moment soil unit-level estimates of cone factor.

5 INVESTIGATION ON SPATIAL VARIABILITY OF NET CONE RESISTANCE

The complexity of depositional and modification processes by which in-situ soil masses are formed

and modified would most likely result in a more or less gradual transition in soil nature (and, hence,

in measured properties) between the stratigraphic units, in accordance with Linnaeus' observation by

which "natura non fecit saltus". It is to be expected that soil properties would generally vary gradually,

both in the horizontal and the vertical direction. This

implies that properties measured at adjacent

spatial locations would generally exhibit stronger correlation than measurements taken at spatially distant

locations. Investigating such phenomena in a quantitative manner requires the application of adequate

techniques, mainly making use of the hypothesis by which soil properties can be modelled as random

variates with an anisotropic spatial correlation structure which is dependent of many factors (e.g. Uzielli

et al. 2006). In the context of second-moment uncertainty characterisation, the investigation of spatial variability

is important because it allows to quantify the effects of spatial averaging on aleatory uncertainty. It

is intuitive that the magnitude of uncertainty would depend on the relative magnitudes of correlation

distance and the spatial extension under investigation. The aleatory uncertainty in a soil property over a

spatial extension L can be estimated, for instance, using the following approximation (Vanmarcke 1983):

in which COV_a is a term representative of aleatory uncertainty and accounting for spatial averaging

effects due to variance reduction; COV_ξ is the coefficient of variation of inherent variability for “point”

properties; and δ is the scale of fluctuation (defined in the following) along the spatial direction of

interest. A more detailed insight into variance reduction is provided in Uzielli et al. (2006).

Figure 19. Example calculation of net cone resistance and results of decomposition procedure for CPTU103.

In the previous sections of this paper, COV_ξ was used as a reference parameter for aleatory uncer

tainty. This is equivalent to setting $\delta/L = 1$ in Eq. (50), and provides a conservative estimate of aleatory

uncertainty in those cases in which spatial correlation may not be investigated.

Here, the spatial correlation structure of q_{net} is investigated in the second-moment sense. This involves

the estimation of at least the scale of fluctuation and the coefficient of variation of inherent variability

on a set of weakly stationary, zero-mean residuals obtained by subtracting a deterministic trend from q_{net}

data. As discussed in greater detail in Uzielli et al. (2006), the scale of fluctuation provides a quantitative

estimate of the spatial extent in which the fluctuations of a property around a deterministic trend are

significantly correlated. The coefficient of variation of inherent variability quantifies the scatter in the

set of residuals.

Due to the considerable anisotropy of soils, it is paramount to specify the reference spatial direction.

Previous studies, cited in Uzielli et al. (2006), have shown that, for a given soil property, the scale of fluctuation in the horizontal direction can exceed that in the vertical direction by several orders of magnitude.

Here, only vertical spatial correlation is addressed. The vertical scale of fluctuation is denoted by δ_v .

Here, only vertical spatial correlation is addressed. The vertical scale of fluctuation is denoted by δ_v .

In Figure 19, an example of the procedure leading to the calculation of net cone resistance from q_c

and u_2 and to the decomposition which is preliminary to the estimation of δ_v and COV ξ is illustrated for

CPTU103: (a) measured cone tip resistance q_c ; (b) measured pore pressure u_2 ; (c) calculated corrected

cone resistance q_t ; (d) estimated total vertical overburden stress; (e) calculated net cone resistance q_{net} ;

(f) trend for q_{net} ; (g) residuals for q_{net} .

Vanmarcke's method is illustrated in greater detail in

Uzielli et al.

(2006).

Once again, the lowest-degree polynomial trend which generates statistically independent residuals

is identified as candidate trend function. Table 16 reports, for each of the 5 available CPTU soundings:

(a) the equation of the deterministic trend obtained by regression; (b) the vertical scale of fluctuation as

estimated by Vanmarcke's procedure; and (c) the coefficient of variation of inherent variability.

The average value of the vertical scales of fluctuation, rounded to the nearest cm, is 0.06 m; the

standard deviation is 0.01 m and the coefficient of variation is 0.18. The corresponding values for the

coefficient of variation of inherent variability are incidentally identical (though dimensionless): 0.06,

0.01 and 0.18, respectively. The low values of COV ω suggest that the average value of the scale of

fluctuation may be used with confidence.

As reported in Uzielli et al. (2006), previous research has shown that estimate values of the scale of

fluctuation are dependent on a number of factors, such as the scale of investigation, the type of trend

Table 16. Summary of spatial variability analysis on CPTU profiles (q net in MPa; z in m).

CPTU Equation of trend in depth range 0.0–15.0 m δv (m)
COV ω

CPTU101 t q net (z) = 0.005z² + 0.0179z + 0.0717 0.08 0.07

CPTU102 t q net (z) = 0.0311z + 0.0374 0.05 0.05

CPTU103 t q net (z) = 0.0011z² + 0.0122z + 0.0889 0.06 0.07

CPTU104 t q net (z) = 0.0300z + 0.0791 0.07 0.07

CPTU105 t q net (z) = 0.0370z+ 0.0287 0.06 0.07

removed during decomposition and the measurement interval of the source data. Hence, while there is

no "correct" value, it is paramount to specify such information when providing numerical estimates of

second-moment spatial correlation parameters.

6 UNCERTAINTY IN UNDRAINED SHEAR STRENGTH CALCULATED FROM CPTU

The fourth phase consists in the second-moment characterisation of the uncertainty in undrained shear

strength in the spatial extension 5.0-15.0 m from seabed. This exercise makes use of the results obtained in

the previous phases: aleatory uncertainty, statistical estimation uncertainty and measurement uncertainty

are taken from the results of second-moment characterisation; transformation uncertainty is provided by

the results of FOSM modeling of the cone factor; variance reduction in aleatory uncertainty is quantified

from the results of random field modeling of net cone resistance. It should be observed that spatially averaged design value of undrained shear strength cannot be

obtained from the values deriving from CAUC and DSS testing, as the scale of fluctuation cannot be

calculated from such data because measurement depth interval is variable and excessively large. Hence,

s ucPTU-CAUC will be calculated from the results of any one CPTU soundings (say, CPTU 101) using

Eq. (32). The following additive model for the total uncertainty in undrained shear strength calculated from

CAUC testing is proposed:

in which COV SP (s ucPTU-CAUC) is the total coefficient of variation of undrained shear strength, accounting

for spatial averaging effects; δv is the vertical scale of

fluctuation of net cone resistance; L is the spatial extension of interest; $COV_{\xi}(q_{net})$ is the coefficient of variation of aleatory uncertainty of net cone resistance; $COV_m(q_{net})$ is the coefficient of variation of measurement uncertainty of q_{net} ; $COV_{SE}(q_{net})$ is the statistical estimation uncertainty related to the decomposition of q_{net} data; and $COV(N_{ktCAUC,UN})$ is the soil unit-level coefficient of variation of transformation uncertainty of the cone factor. If CPTU 101 is taken as reference sounding, decomposition yields the following trend function (q_{net}

in MPa; z in m):

It should be noted that such trend is different from the one reported for CPTU101 in Table 16, as regression

has been performed on a reduced data set (i.e. only the range 5.0-15.0 m), resulting in different best-fit

parameters. The nominal value of undrained shear strength is calculated as:

in which $\mu[t(q_{net})]$ is the mean value of $t(q_{net})$ in the spatial extension of interest. Following decomposition

and application of Vanmarcke's method, the scale of fluctuation is estimated to be $\delta_v = 0.06$ m and the

coefficient of variation of inherent variability is $COV_{\xi}(q_{net}) = 0.05$. It should be noted that these values

are different from the ones obtained for CPTU101 if the entire depth range is considered; this shows

that the scale of fluctuation is not an intrinsic parameter of a soil unit. In the present case, the reference

spatial extension is $L = 10.0$ m. The coefficient of variation of measurement uncertainty $COV_m(q_{net})$

cannot be estimated directly as q_{net} is a derived parameter; as the sources of measurement error in q_c , u_2

and p_0 can be assumed to be mutually uncorrelated (operator/procedural error is very limited in CPTU

testing), the following additive model is proposed:

in which, as seen previously, $COV_m(q_c) = 0.07$ and $COV_m(u_2) = 0.07$. The coefficient of variation of

measurement error of total overburden stress is that of submerged unit weight; hence $COV_m(p_0) = 0.03$.

From Eq. (54), $COV_m(q_{net}) = 0.10$.

Using Eq. (12), and considering that the numerosity of the reduced CPTU101 sounding is $n_s = 1000$,

the coefficient of variation of statistical estimation error $COV_{SE}(q_{net})$ can be estimated at 0.002; lastly,

the coefficient of variation of transformation uncertainty is $COV(N_{kt,UN}) = 0.29$ as obtained previously.

Applying Eq. (51), the total coefficient of variation for spatially averaged undrained shear strength is

$COV(s_{uCPTU-CAUC,SP}) = 0.31$. It may be seen that, due to spatial averaging, the aleatory uncertainty term

has very reduced influence on total uncertainty; moreover, the high numerosity of the CPTU sounding

yields very low statistical estimation uncertainty. Measurement error and - to a much larger extent -

transformation uncertainty determine the total level of uncertainty according to the proposed model.

It is of interest to compare the above result with the case in which spatial averaging effects

are not considered. Setting $\delta v/L = 1$ in Eq. (51), the total coefficient of variation is again

$COV(s_{uCPTU-CAUC,SP}) = 0.31$. Hence, it is seen that, in this example, transformation uncertainty and

measurement uncertainty still dominate overall uncertainty. In many cases, however, neglecting spatial

averaging can lead to substantial overestimation of uncertainty.

It should be noted that COV (s uCPTU-CAUC,SP) is representative of the entire spatial extension under

investigation. As decomposition is performed by regression, it is implicitly assumed that the standard

deviation of the residuals of decomposition would be constant throughout the same spatial extension. This

would in turn imply that the trend function is at least approximately constant; hence, the nominal value

given by Eq. (53) should be regarded as an "average" value over the spatial extension. If a different spatial

extension were of interest, re-calculation of several of the components of total uncertainty - namely:

trend, inherent variability, statistical estimation uncertainty and spatial averaging effects - would be

required.

7 BAYESIAN UPDATING OF UNDRAINED SHEAR STRENGTH

In the previous sections of the paper, the classical statistical approach was used to estimate distribution

parameters in the second-moment sense. Such an approach assumes that the parameters are unknown

constants, and that sample statistics may be calculated to estimate them. It has been highlighted that

estimators are invariably imperfect due to the epistemic uncertainties in sample data and limited sample

size. It has been shown that a reduction in statistical estimation error can be achieved by increasing

sample size, i.e. by obtaining more measurements; however, in geotechnical engineering applications,

this is generally not feasible in practice. Statistical estimates must frequently be integrated by subjective

information, engineering judgement or other data sets obtained using different measurement techniques.

While the classical statistical approach does not allow for

the combination of judgmental (subjective) and observational (objective) data, or the merging of data from different sources (e.g. different measurement techniques), the Bayesian perspective may be referred to if such integration is of interest.

In the Bayesian approach, the unknown parameters of a distribution are also assumed to be random variables, and modelled as such. Thus, the uncertainties in distinct sets of parameters can be combined formally using Bayes' theorem (e.g. Ang & Tang 1975), yielding an integrated estimation.

In a previous section, a second-moment estimate of the cone factor N_{ktCAUC} was obtained for each

CAUC measurement of s_{uCAUC} using a representative value of q_{net} at the same nominal depth. In this

section, the Bayesian approach is adopted for the estimation of undrained shear strength from a com

bination of CAUC measurements and CPTU data. It is perhaps more pertinent to refer to "Bayesian

updating" in the sense that the information from one data set is used to update the information from the

other and obtain an integrated estimate.

Here, Bayesian updating is performed for each of the $n_s = 17$ CAUC measurements s_{uCAUC} . Second

moment parameters of each CAUC measurement of s_{uCAUC} are used as prior information. The second

moment parameters of the $s_{uCPTU-CAUC}$ values obtained from CPTU data at the same nominal depth of

Table 17. Calculation of second-moment parameters of s_{uCAUC} . $\mu_{s(q_{net})}$ $\sigma_{s(q_{net})}$ $\mu(s_{uCPTU-CAUC})$ $\sigma(s_{uCPTU-CAUC})$

j [kPa] [kPa] $\mu(N_{ktCAUC})$ $\sigma(N_{ktCAUC})$ [kPa] [kPa]

1 208.1 35.5 15.0 3.0 13.9 3.6

2	216.0	33.2	15.0	3.0	14.4	3.6
3	221.1	30.7	15.0	3.0	14.7	3.6
4	238.5	34.3	15.0	3.0	15.9	3.9
5	258.4	40.0	15.0	3.0	17.2	4.4
6	273.7	45.7	15.0	3.0	18.2	4.8
7	304.6	32.3	15.0	3.0	20.3	4.6
8	344.4	44.9	15.0	3.0	23.0	5.5
9	364.0	48.5	15.0	3.0	24.3	5.8
10	390.9	47.5	15.0	3.0	26.1	6.1
11	400.1	46.5	15.0	3.0	26.7	6.2
12	423.5	53.9	15.0	3.0	28.2	6.7
13	435.0	56.3	15.0	3.0	29.0	6.9
14	435.0	56.3	15.0	3.0	29.0	6.9
15	466.3	48.4	15.0	3.0	31.1	7.0
16	471.9	39.0	15.0	3.0	31.5	6.8
17	476.4	30.6	15.0	3.0	31.8	6.7

each CAUC measurement are taken to be the parameters of the likelihood function which is used to

update the prior information and obtain a posterior estimate, also in the second-moment sense. Consistently with the previous statistical analyses in the paper, it is assumed that all distributions are

Gaussian (normal). The mean of the posterior distribution of undrained shear strength at the j -th nominal

depth is then given by

while the standard deviation is

in which $\mu(s_{UCPTU-CAUC})$ and $\sigma(s_{UCPTU-CAUC})$ are the mean and standard deviation of the set of $n_{c,j}$ values

of $s_{u\text{CPTU-CAUC}}$ calculated from q_{net} at the j -th nominal depth; $\mu(s_{u\text{CAUC}})$ and $\sigma(s_{u\text{CAUC}})$ are the mean (i.e.

the measured value) and standard deviation of the CAUC-measured undrained shear strength; and $n_{c,j}$ is

the number of $s_{u\text{CPTU-CAUC}}$ values at the j -th nominal depth as defined in section 3.4 (in this case $n_{c,j} = 1$

at every nominal depth). To apply Eq. (55) and Eq. (56), the values of $s_{u\text{CPTU-CAUC}}$ at each nominal depth must be calculated.

To do this, it is necessary to refer to Eq. (32). Recommended values of $N_{kt\text{CAUC}}$ have been provided by

NGI (1989) for Troll clay. In this exercise of Bayesian updating, however, such values will be neglected

in favor of a fully subjective initial value, as to simulate a lack of objective information. On the basis

of the literature review produced by Lunne et al. (1997), a mean value $\mu(N_{kt\text{CAUC}}) = 15.0$ is selected.

A "medium-high" coefficient of variation $\text{COV}(N_{kt\text{CAUC}}) = 0.20$ is also taken. The underlined "N" is

used to distinguish the "subjective" cone factor from the "objective" one calculated in section 4. The

standard deviation of the subjective cone factor is thus $\sigma(N_{kt\text{CAUC}}) = 3.0$. FOSM approximation of a ratio of variates [see Eq. (35) and Eq. (36)] can now be used to calculate

second-moment parameters of $s_{u\text{CPTU-CAUC}}$ using Eq. (55) and Eq. (56). The second-moment parameters

$[\mu_{s,j}(q_{\text{net}})]$ and $\sigma_{s,j}(q_{\text{net}})$ for q_{net} are calculated as explained in section 4 by Eq. (38) and Eq. (41), respec

tively, and shown in Figure 18b. Table 17 reports the results of FOSM approximation of $s_{u\text{CPTU-CAUC}}$. Eq. (55) and Eq. (56). can now be applied to obtain second-moment parameters of the "updated"

undrained shear strength s_{uB} . The results of Bayesian

updating are shown in Table 18. It may be observed that CAUC-measured values are higher than CPTU-derived values at shallow

depths, and that the opposite is true at greater depths in UN1. This suggests that although the subjective

estimated mean value of 15 can be adequate, N_{ktCAUC} should vary with depth in UN1, is probably greater

Table 18. Calculation of second-moment parameters of s_{uB} by Bayesian updating. $\mu(s_{uCAUC})$ $\sigma(s_{uCAUC})$ $\mu(s_{uCPTU-CAUC})$ $\sigma(s_{uCPTU-CAUC})$ $\mu(s_{uB})$ $\sigma(s_{uB})$

j [kPa] [kPa] [kPa] [kPa] [kPa] [kPa]

1 12.2 2.4 13.9 3.6 12.7 2.0

2 14.8 3.0 14.4 3.6 14.6 2.3

3 13.3 2.7 14.7 3.6 13.8 2.1

4 15.1 3.0 15.9 3.9 15.4 2.4

5 18.2 3.6 17.2 4.4 17.8 2.8

6 20.5 4.1 18.2 4.8 19.5 3.1

7 22.0 4.4 20.3 4.6 21.2 3.2

8 22.8 4.6 23.0 5.5 22.9 3.5

9 24.9 5.0 24.3 5.8 24.6 3.8

10 27.5 5.5 26.1 6.1 26.9 4.1

11 32.9 6.6 26.7 6.2 29.6 4.5

12 28.8 5.8 28.2 6.7 28.6 4.4

13 32.9 6.6 29.0 6.9 31.0 4.8

14 33.1 6.6 29.0 6.9 31.1 4.8

15 38.6 7.7 31.1 7.0 34.5 5.2

16 37.0 7.4 31.5 6.8 34.0 5.0

17 44.5 8.9 31.8 6.7 36.3 5.3

than 15 at shallow depths and smaller than 15 at greater depths; such trend is consistent with the findings

in section 4 shown in Figure 18c. It is important to note that, for each calculation instance in Table 18 the

standard deviation of s_{uB} is smaller than those of s_{uCAUC} and $s_{uCPTU-CAUC}$. This is a general, beneficial

result of Bayesian updating, which is significant for hazard assessment as it yields a lower uncertainty.

8 CONCLUSIONS

Uncertainty-based second-moment geotechnical characterisation of the two top soil units from the Troll

clay site was performed. Second-moment statistics were obtained from laboratory and in-situ test meas

urements. First-order second-moment approximation (FOSM) was used to characterize a number of

derived parameters in the second-moment sense. The spatial variability of net cone resistance was inves

tigated for the upper layer. Characterisation of the uncertainty of undrained shear strength (calculated

from CPTU data) was performed, and the relative magnitude of the uncertainty components was dis

cussed. As a conclusive exercise, Bayesian updating was used to combine the values of undrained shear

strength resulting from triaxial compression tests and piezocone tests.

Results of uncertainty-based characterisation are in good accordance with previous deterministic site

characterisation efforts by NGI (1989). In several cases, it was observed that the main contributing

sources to total uncertainty were measurement uncertainty and transformation uncertainty. Hence, it is

paramount to report the testing procedure used to obtain a given set of measurements (and, if available,

specific values of measurement uncertainty), as well as the models used to derive engineering parameters

(e.g. undrained shear strength).

It should be stressed that the results obtained for the Troll data (including values of uncertainty components

and total uncertainty) should not be uncritically exported to data from other sites. The components

of uncertainty depend on factors (e.g. trend functions) which may be strictly site or case-specific. Site

specificity has no bearing on the uncertainty-based characterisation methods applied herein; rather, the

hypotheses adopted, as well as intermediate results, should be clearly stated, and presented along with

the output uncertainty estimates.

Lastly - and perhaps most importantly - engineering judgment and geotechnical expertise are essential

in every phase of uncertainty-based characterisation: in the preliminary examination of data; in the critical

evaluation of second-moment statistics of measured data; in the formulation of appropriate uncertainty

models for each parameter and in the selection of appropriate transformation models to obtain parameters

useful for design.

Given the increasing importance of uncertainty estimation in geotechnical design codes, it is hoped

that the present paper can serve as a reference and stimulus for future similar uncertainty-based

characterisation efforts.

ACKNOWLEDGEMENTS

The authors are grateful to Serge Leroueil for suggesting the topic of the paper. The patient and skilful

cooperation of Erik Lyche Solheim in the retrieval of data

by digitisation is gratefully acknowledged, as

well as the kind availability of Rune Dyvik in discussing the nature and implications of measurement

error in triaxial testing. Statoil, represented by Tor Inge Tjelta, kindly allowed utilisation of the CPTU

data from the 2005 investigations. The authors wish to thank the International Centre for Geohazards

(ICG) and the Research Council of Norway for supporting the work presented in this paper.

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